SECOND EDITION

ENGINEERING GEOLOGY

F. G. BELL

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Second Edition

F. G. Bell

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Preface

As noted in the Preface to the first edition, engineering geology can be defined as the application of Geology to engineering practice. In other words, it is concerned with those geological factors that influence the location, design, construction and maintenance of engineering works. Accordingly, it draws on a number of geological disciplines such as geomorphology, structural geology, sedimentology, petrology and stratigraphy. In addition, engineering geology involves hydrogeology and some understanding of rock and soil mechanics.

Similar to the first edition, this edition too is written for undergraduate and post-graduate students of engineering geology. It is hoped that this will also be of value to those involved in the profession, especially at the earlier stages of their careers. However, it is aimed at not just engineering geologists but also at those in civil and mining engineering, water engineering, quarrying and, to a lesser extent, architecture, planning, surveying and building. In other words, those who deal with the ground should know something about it.

No single textbook can cover all the needs of the variety of readers who may use it. Therefore, a list of books is suggested for further reading, and references are provided for those who want to pursue some aspect of the subject matter to greater depth. However, some background knowledge also is assumed. Obviously, students of geology will have done much more reading on geology than the basic geological material covered in this book. They presumably will have done or will do some reading on soil mechanics and rock mechanics. On the other hand, those with an engineering background will have read some soil and rock mechanics, but need some basic geology, hopefully, this book will meet their needs. Moreover, any book will reflect the background of its author and his or her view of the subject. However, this author has attempted to give a balanced overview of the subject.

The text has been revised and extended to take account of some subjects that were not dealt with in the first edition. Also, some of the chapters have been rearranged. Hopefully, this should have improved the text.

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Rock Types and Stratigraphy

According to their origin, rocks are divided into three groups, namely, the igneous, metamorphic and sedimentary rocks.

Igneous Rocks

Igneous rocks are formed when hot molten rock material called magma solidifies. Magmas are developed when melting occurs either within or beneath the Earth's crust, that is, in the upper mantle. They comprise hot solutions of several liquid phases, the most conspicuous of which is a complex silicate phase. Thus, igneous rocks are composed principally of silicate minerals. Furthermore, of the silicate minerals, six families – the olivines [$(Mg,Fe)_2SiO_4$], the pyroxenes [e.g. augite, $(Ca, Mg, Fe, Al)_2(Al,Si)_2O_6$], the amphiboles [e.g. hornblende, $(Ca,Na,Mg,Fe,Al)_{7\text{-}8}(Al,Si)_8O_{22}(OH)_2$], the micas [e.g. muscovite, $KAl_2(AlSi_2)_{10}(O,F)_2$; and biotite, $K(Mg,Fe)_2(AlSi_3)O_{10}(OH,F)_2$], the feldspars (e.g. orthoclase, $KAlSi_3O_8$; albite, $NaAlSi_3O_8$; and anorthite, $CaAl_2Si_2O_8$) and the silica minerals (e.g. quartz, SiO_2) – are quantitatively by far the most important constituents. Figure 1.1 shows the approximate distribution of these minerals in the commonest igneous rocks.

Igneous rocks may be divided into intrusive and extrusive types, according to their mode of occurrence. In the former type, the magma crystallizes within the Earth's crust, whereas in the latter, it solidifies at the surface, having erupted as lavas and/or pyroclasts from a volcano. The intrusions have been exposed at the surface by erosion. They have been further subdivided on the basis of their size, that is, into major (plutonic) and minor (hypabyssal) categories.

Igneous Intrusions

The form that intrusions adopt may be influenced by the structure of the host or country rocks. This applies particularly to minor intrusions.

Dykes are discordant igneous intrusions, that is, they traverse their host rocks at an angle and are steeply dipping (Fig. 1.2). As a consequence, their surface outcrop is little affected by topography and, in fact, they tend to strike a straight course. Dykes range in width up to

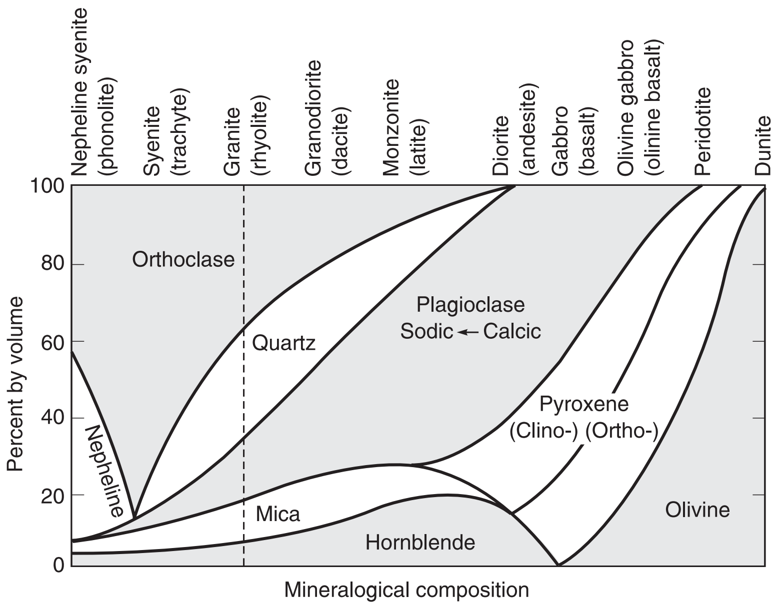


Figure 1.1

Approximate mineral compositions of the more common types of igneous rocks, e.g. granite approximately 40% orthoclase, 33% quartz, 13% plagioclase, 9% mica and 5% hornblende (plutonic types without brackets, volcanic equivalents in brackets).

Figure 1.2

Dyke on the south side of the Isle of Skye, Scotland.

several tens of metres but their average width is on the order of a few metres. The length of their surface outcrop also varies; for example, the Cleveland Dyke in the north of England can be traced over some 200 km. Dykelets may extend from and run parallel to large dykes, and irregular offshoots may branch away from large dykes. Dykes do not usually have an upward termination, although they may have acted as feeders for lava flows and sills. They often occur along faults, which provide a natural path of escape for the injected magma. Most dykes are of basaltic composition. However, dykes may be multiple or composite. Multiple dykes are formed by two or more injections of the same material that occur at different times. A composite dyke involves two or more injections of magma of different composition.

Sills, like dykes, are parallel-sided igneous intrusions that can occur over relatively extensive areas. Their thickness, however, can vary. Unlike dykes, they are injected in an approximately horizontal direction, although their attitude may be subsequently altered by folding. When sills form in a series of sedimentary rocks, the magma is injected along bedding planes (Fig. 1.3). Nevertheless, an individual sill may transgress upwards from one horizon to another. Because sills are intruded along bedding planes, they are said to be concordant, and their outcrop is similar to that of the host rocks. Sills may be fed from dykes, and small dykes

Figure 1.3

The Whin Sill, Northumberland, England.

may arise from sills. Most sills are composed of basic igneous material. Sills may also be multiple or composite in character.

The major intrusions include batholiths, stocks and bosses. Batholiths are very large in size and are generally of granitic or granodioritic composition. Indeed, many batholiths have an immense surface exposure. For instance, the Coast Range batholith of Alaska and British Columbia can be traced over 1000 km in length and over approximately 130 to 190 km in width. Batholiths are associated with orogenic regions. They often appear to have no visible base, and their contacts are well-defined and dip steeply outwards. Bosses are distinguished from stocks in that they have a more or less circular outcrop. Both their surface exposures are of limited size, frequently less than 100 km^2. They may represent upward extensions from deep-seated batholiths.

Certain structures are associated with granite massifs, tending to be best developed at the margins. For example, particles of elongate habit may be aligned with their long axes parallel to each other. Most joints and minor faults in batholiths possess a relationship with the shape of the intrusion. Fractures are developed in the solidified margins of a plutonic mass and may have been filled with material from the interior when it was still liquid. Cross joints or Q joints tend to radiate from the centre of the massif. They are crossed approximately at right angles by steeply dipping joints termed longitudinal or S joints. Pegmatites or aplites (see the following text) may be injected along both types of joints mentioned. Diagonal joints are orientated at 45° to Q and S joints. Flat-lying joints may be developed during or after formation of the batholith and they may be distinguished as primary and secondary, respectively. Normal faults and thrusts occur in the marginal zones of large intrusions and the adjacent country rocks.

Volcanic Activity and Extrusive Rocks

Volcanic zones are associated with the boundaries of the crustal plates (Fig. 1.4). Plates can be largely continental, oceanic, or both. Oceanic crust is composed of basaltic material, whereas continental crust varies from granitic in the upper part to basaltic in the lower. At destructive plate margins, oceanic plates are overridden by continental plates. The descent of the oceanic plate, together with any associated sediments, into zones of higher temperature leads to melting and the formation of magmas. Such magmas vary in composition, but some, such as andesitic or rhyolitic magma, may be richer in silica, which means that they are more viscous and, therefore, do not liberate gas so easily. The latter type of magmas are often responsible for violent eruptions. In contrast, at constructive plate margins, where plates are diverging, the associated volcanic activity is a consequence of magma formation in the lower crust or upper mantle. The magma is of basaltic composition, which is less

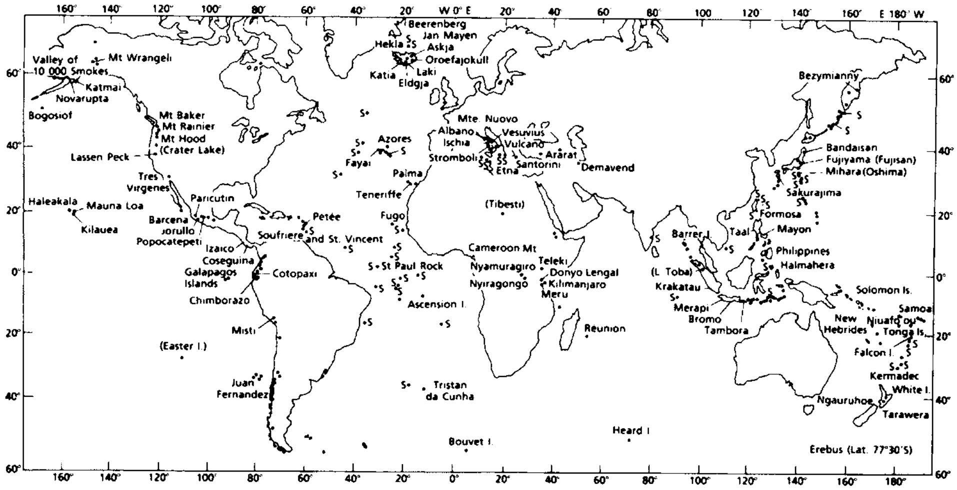


Figure 1.4

Distribution of the active volcanoes in the world. S, submarine eruptions.

viscous than andesitic or rhyolitic magma. Hence, there is relatively little explosive activity and the associated lava flows are more mobile. However, certain volcanoes, for example, those of the Hawaiian Islands, are located in the centres of plates. Obviously, these volcanoes are unrelated to plate boundaries. They owe their origins to hot spots in the Earth's crust located above rising mantle plumes. Most volcanic material is of basaltic composition.

Volcanic activity is a surface manifestation of a disordered state within the Earth's interior that has led to the melting of material and the consequent formation of magma. This magma travels to the surface, where it is extravasated either from a fissure or a central vent. In some cases, instead of flowing from the volcano as lava, the magma is exploded into the air by the rapid escape of the gases from within it. The fragments produced by explosive activity are known collectively as pyroclasts.

Eruptions from volcanoes are spasmodic rather than continuous. Between eruptions, activity may still be witnessed in the form of steam and vapours issuing from small vents named fumaroles or solfataras. But, in some volcanoes, even this form of surface manifestation ceases, and such a dormant state may continue for centuries. To all intents and purposes, these volcanoes appear extinct. In old age, the activity of a volcano becomes limited to emissions of gases from fumaroles and hot water from geysers and hot springs.

Steam may account for over 90% of the gases emitted during a volcanic eruption. Other gases present include carbon dioxide, carbon monoxide, sulphur dioxide, sulphur trioxide, hydrogen sulphide, hydrogen chloride and hydrogen fluoride. Small quantities of methane, ammonia, nitrogen, hydrogen thiocyanate, carbonyl sulphide, silicon tetrafluoride, ferric chloride, aluminium chloride, ammonium chloride and argon have also been noted in volcanic gases. It has often been found that hydrogen chloride is, next to steam, the major gas produced during an eruption but that the sulphurous gases take over this role in the later stages.

At high pressures, gas is held in solution, but as the pressure falls, gas is released by the magma. The rate at which it escapes determines the explosivity of the eruption. An explosive eruption occurs when, because of its high viscosity (to a large extent, the viscosity is governed by the silica content), the magma cannot readily allow the escape of gas until the pressure that it is under is lowered sufficiently to allow this to occur. This occurs at or near the surface. The degree of explosivity is only secondarily related to the amount of gas the magma holds. On the other hand, volatiles escape quietly from very fluid magmas.

Pyroclasts may consist of fragments of lava that were exploded on eruption, of fragments of pre-existing solidified lava or pyroclasts, or of fragments of country rock that, in both latter instances, have been blown from the neck of a volcano.

The size of pyroclasts varies enormously. It is dependent on the viscosity of the magma, the violence of the explosive activity, the amount of gas coming out of solution during the flight of the pyroclast, and the height to which it is thrown. The largest blocks thrown into the air may weigh over 100 tonnes, whereas the smallest consist of very fine ash that may take years to fall back to the Earth's surface. The largest pyroclasts are referred to as volcanic bombs. These consist of clots of lava or of fragments of wall rock.

The term lapilli is applied to pyroclastic material that has a diameter varying from approximately 10 to 50 mm (Fig. 1.5). Cinder or scoria is irregular-shaped material of lapilli size. It usually is glassy and fairly to highly vesicular.

The finest pyroclastic material is called ash. Much more ash is produced on eruption of acidic than basic magmas. Acidic igneous rocks contain over 65% silica, whereas basic igneous rocks contain between 45 and 55%. Those rocks that have a silica content between acid and basic are referred to as intermediate, and those with less than 45% silica are termed ultrabasic. As mentioned, the reason for the difference in explosivity is because acidic material is more viscous than basic or basaltic lava.

Beds of ash commonly show lateral variation as well as vertical. In other words, with increasing distance from the parent vent, the ash becomes finer and, in the second case, because the heavier material falls first, ashes frequently exhibit graded bedding, with coarser material occurring at the base of a bed, and becoming finer towards the top. Reverse grading may

Figure 1.5

Lapilli near Crater Lake caldera, Oregon.

occur as a consequence of an increase in the violence of eruption or changes in wind velocity. The spatial distribution of ash is influenced by wind direction, and deposits on the leeward side of a volcano may be much more extensive than on the windward. Indeed, they may be virtually absent from the latter side.

After pyroclastic material has fallen back to the ground surface, it eventually becomes indurated. It then is described as tuff. According to the material of which tuff is composed, distinction can be drawn between ash tuff, pumiceous tuff and tuff breccia. Tuffs are usually well bedded, and the deposits of individual eruptions may be separated by thin bands of fossil soil or old erosion surfaces. Pyroclast deposits that accumulate beneath the sea are often mixed with a varying amount of sediment and are referred to as tuffites. Rocks that consist of fragments of volcanic ejectamenta set in a fine-grained groundmass are referred to as agglomerate or volcanic breccia, depending on whether the fragments are rounded or angular, respectively.

When clouds or showers of intensely heated, incandescent lava spray fall to the ground, they weld together to become welded tuff. In other cases, because the particles become intimately fused with each other, they attain a largely pseudo-viscous state, especially in the deeper

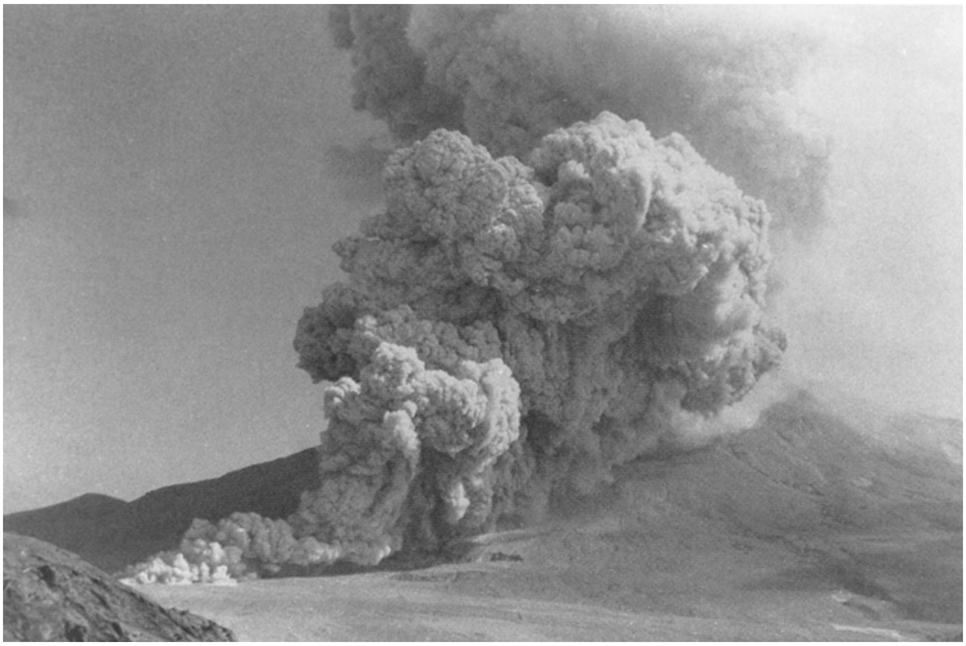


Figure 1.6

Nuee ardente erupting from Mt. St. Helens in May 1980, Washington State.

parts of the deposit. The term ignimbrite is used to describe these rocks. If ignimbrites are deposited on a steep slope, they begin to flow, and they resemble lava flows. Ignimbrites are associated with nuées ardentes (Fig. 1.6).

Lavas are emitted from volcanoes at temperatures only slightly above their freezing points. During the course of their flow, the temperature falls until solidification takes place somewhere between 600 and 900°C, depending on their chemical composition and gas content. Basic lavas solidify at higher temperatures than do acidic ones.

Generally, flow within a lava stream is laminar. The rate of flow of lava is determined by the gradient of the slope down which it moves and by its viscosity that, in turn, is governed by its composition, temperature and volatile content. Because of their lower viscosity, basic lavas flow much faster and further than do acid lavas. Indeed, the former type has been known to travel at speeds of up to 80 km h^{-1}.

The upper surface of a recently solidified lava flow develops a hummocky, ropy (termed pahoehoe); rough, fragmental, clinkery, spiny (termed aa); or blocky structure (Fig. 1.7a and b). The pahoehoe is the most fundamental type, however, some way downslope from the vent,

Figure 1.7

(a) Ropy or pahoehoe lava, Craters of the Moon, Idaho.

it may give way to aa or block lava. In other cases, aa or block lava, may be traceable into the vent. The surface of lava solidifies before the main body of the flow beneath. Pipes, vesicle trains or spiracles may be developed in the lava, depending on the amount of gas given off, the resistance offered by the lava and the speed at which it flows. Pipes are tubes that project upwards from the base and are usually several centimetres in length and a centimetre or less in diameter. Vesicles are small spherical openings formed by gas. Vesicle trains form when gas action has not been strong enough to produce pipes. Spiracles are openings formed by explosive disruption of the still-fluid lava by gas generated beneath it. Large flows are fed by a complex of streams beneath the surface crust so that when the supply of lava is exhausted, the stream of liquid may drain away leaving a tunnel behind.

Thin lava flows are broken by joints that may run either at right angles or parallel to the direction of flow. Joints do occur with other orientations but are much less common. Those joints that are normal to the lava surface usually display a polygonal arrangement, but only rarely do they give rise to columnar jointing. These joints develop as the lava cools. First, primary joints form, from which secondary joints arise, and so it continues.

Figure 1.7, cont'd

(b) Clinkery or aa lava, Craters of the Moon, Idaho.

Typical columnar jointing is developed in thick flows of basalt (Fig. 1.8). The columns in columnar jointing are interrupted by cross joints that may be either flat or saucer-shaped. The latter may be convex up or down. These are not to be confused with platy joints that are developed in lavas as they become more viscous on cooling, so that slight shearing occurs along flow planes.

Texture of Igneous Rocks

The degree of crystallinity is one of the most important items of texture. An igneous rock may be composed of an aggregate of crystals, of natural glass, or of crystals and glass in varying proportions. This depends on the rate of cooling and composition of the magma on the one hand and the environment under which the rock developed on the other. If a rock is completely composed of crystalline mineral material, it is described as holocrystalline. Most rocks are holocrystalline. Conversely, rocks that consist entirely of glassy material are referred to as holohyaline. The terms hypo-, hemi- or merocrystalline are given to rocks that are made up of intermediate proportions of crystalline and glassy materials.

Figure 1.8

Columnar jointing in basalt, Giant's Causeway, Northern Ireland.

When referring to the size of individual crystals, they are described as cryptocrystalline if they can just be seen under the highest resolution of the microscope or as microcrystalline if they can be seen at a lower magnification. These two types, together with glassy rocks, are collectively described as aphanitic, which means that the individual minerals cannot be distinguished with the naked eye. When the minerals of which a rock is composed are mega- or macroscopic, that is, they can be recognized with the unaided eye, it is described as phanerocrystalline. Three grades of megascopic texture are usually distinguished, namely, fine-grained, medium-grained and coarse-grained, the limits being under 1-mm diameter, between 1- and 5-mm diameter, and over 5-mm diameter, respectively.

A granular texture is one in which there is no glassy material and the individual crystals have a grain-like appearance. If the minerals are of approximately the same size, the texture is described as equigranular, whereas if this is not the case, it is referred to as inequigranular. Equigranular textures are more typically found in plutonic igneous rocks. Many volcanic rocks and rocks that occur in dykes and sills, in particular, display inequigranular textures, the most important type being the porphyritic texture. In this texture, large crystals or phenocrysts are set in a fine-grained groundmass. A porphyritic texture may be distinguished as macro- or micro-porphyritic, according to whether or not it may be observed with the unaided eye, respectively.

The most important rock-forming minerals are often referred to as felsic and mafic, depending on whether they are light or dark coloured, respectively. Felsic minerals include quartz, muscovite, feldspars and feldspathoids, whereas olivines, pyroxenes, amphiboles and biotite are mafic minerals. The colour index of a rock is an expression of the percentage of mafic minerals that it contains. Four categories have been distinguished:

(1) leucocratic rocks, which contain less than 30% dark minerals
(2) mesocratic rocks, which contain between 30 and 60% dark minerals
(3) melanocratic rocks, which contain between 60 and 90% dark minerals and
(4) hypermelanic rocks, which contain over 90% dark minerals

Usually, acidic rocks are leucocratic, whereas basic and ultrabasic rocks are melanocratic and hypermelanic, respectively.

Igneous Rock Types

Granites and granodiorites are the commonest rocks of the plutonic association. They are characterized by a coarse-grained, holocrystalline, granular texture. Although the term granite lacks precision, normal granite has been defined as a rock in which quartz forms more than 5% and less than 50% of the quarfeloids (quartz, feldspar, feldspathoid content), potash feldspar constitutes 50 to 95% of the total feldspar content, the plagioclase is sodi-calcic, and the mafites form more than 5% and less than 50% of the total constituents (Fig. 1.1).

In granodiorite, the plagioclase is oligoclase or andesine and is at least double the amount of potash feldspar present, the latter forming 8 to 20% of the rock. The plagioclases are nearly always euhedral (minerals completely bounded by crystal faces), as may be biotite and hornblende. These minerals are set in a quartz–potash feldspar matrix.

The term pegmatite refers to coarse or very-coarse-grained rocks that are formed during the last stages of crystallization from a magma. Pegmatitic facies, although commonly associated with granitic rocks, are found in association with all types of plutonic rocks. Pegmatites occur as dykes, sills, veins, lenses or irregular pockets in the host rocks, with which they rarely have sharp contacts (Fig. 1.9).

Aplites occur as veins, usually several tens of millimetres thick, in granites, although like pegmatites they are found in association with other plutonic rocks. They possess a fine-grained, equigranular texture. There is no important chemical difference between aplite and pegmatite, and it is assumed that they both have crystallized from residual magmatic solutions.

Figure 1.9

Pegmatite vein cutting through Shap Granite, Cumbria, England.

Rhyolites are acidic extrusive rocks that are commonly associated with andesites. They are generally regarded as representing the volcanic equivalent of granite. They are usually leucocratic and sometimes exhibit flow banding. Rhyolites may be holocrystalline, but very often they contain an appreciable amount of glass. They are frequently porphyritic, the phenocrysts varying in size and abundance. The phenocrysts occur in a glassy, cryptocrystalline or microcrystalline groundmass. Vesicles are usually found in these rocks.

Acidic rocks occurring in dykes or sills are often porphyritic, quartz porphyry being the commonest example. Quartz porphyry is similar in composition to rhyolite.

Syenites are plutonic rocks that have a granular texture and consist of potash feldspar, a subordinate amount of sodic plagioclase and some mafic minerals, usually biotite or hornblende.

Diorite has been defined as an intermediate plutonic, granular rock composed of plagioclase and hornblende, although at times the latter may be partially or completely replaced by biotite and/or pyroxene. Plagioclase, in the form of oligoclase and andesine, is the dominant feldspar. If orthoclase is present, it acts only as an accessory mineral.

Trachytes and andesites are the fine-grained equivalents of syenites and diorites, respectively. Andesite is the commoner of the two types. Trachytes are extrusive rocks, which are often porphyritic, in which alkali feldspars are dominant. Most phenocrysts are composed of alkali feldspar and, to a lesser extent, of alkali–lime feldspar. More rarely, biotite, hornblende and/or augite may form phenocrysts. The groundmass is usually a holocrystalline aggregate of sanidine (a high-temperature form of orthoclase) laths. Andesites are commonly porphyritic, with a holocrystalline groundmass. Plagioclase (oligoclase-andesine), which is the dominant feldspar, forms most of the phenocrysts. The plagioclases of the groundmass are more sodic than those of the phenocrysts. Sanidine and anorthoclase [$(Na,K)AlSi_3O_8$] rarely form phenocrysts but the former mineral does occur in the groundmass and may encircle some of the plagioclase phenocrysts. Hornblende is the commonest of the ferro-magnesian minerals and may occur as phenocrysts or in the groundmass, as may biotite and pyroxene.

Gabbros and norites are plutonic igneous rocks with granular textures. They are dark in colour. Plagioclase, commonly labradorite, is usually the dominant mineral in gabbros and norites, but bytownite also occurs. The pyroxenes found in gabbros are typically augite, diopsidic augite and diallage. They are usually subhedral (some crystal faces developed) or anhedral (no crystal faces developed). Norites, unlike gabbros, contain orthopyroxenes instead of clinopyroxenes, hypersthene being the principle pyroxene.

Basalts are the extrusive equivalents of gabbros and norites, and are composed principally of calcic plagioclase and pyroxene in roughly equal amounts, or there may be an excess of plagioclase. It is by far the most important type of extrusive rock. Basalts also occur in dykes, cone sheets, sills and volcanic plugs. Basalts exhibit a great variety of textures and may be holocrystalline or merocrystalline, equigranular or macro- or microporphyritic.

Dolerites are found in minor intrusions. They consist primarily of plagioclase, usually labradorite, and pyroxene, usually augite (Fig. 1.10). The plagioclase may occur as phenocrysts, in addition to being one of the principal minerals in the groundmass. Dolerites are fine-to-medium grained. They are usually equigranular but, as they grade towards basalts, they tend to become porphyritic. Nevertheless, phenocrysts generally constitute less than 10% of the rock. The groundmass consists of plagioclase laths, small anhedral pyroxenes and minor amounts of ores.

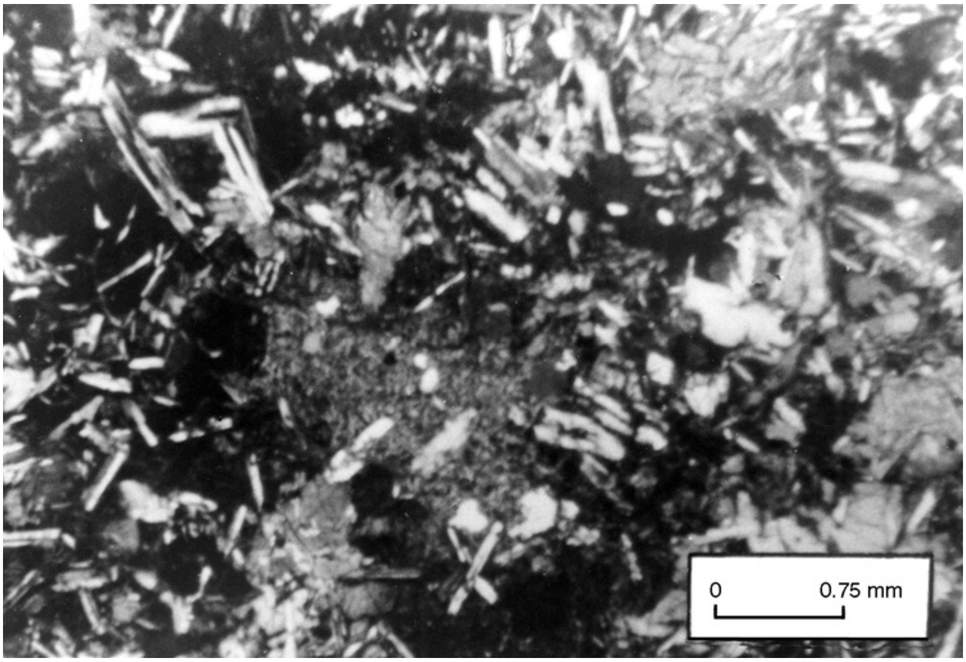


Figure 1.10

Thin section of dolerite from Harrisburg, South Africa, showing patches of clay minerals and some microfracturing.

Metamorphism and Metamorphic Rocks

Metamorphic rocks are derived from pre-existing rock types and have undergone mineralogical, textural and structural changes. These changes have been brought about by changes that have taken place in the physical and chemical environments in which the rocks existed. The processes responsible for change give rise to progressive transformation in rock that takes place in the solid state. The changing conditions of temperature and/or pressure are the primary agents causing metamorphic reactions in rocks. Some minerals are stable over limited temperature–pressure conditions, which means that when these limits are exceeded mineralogical adjustment has to be made to establish equilibrium with the new environment.

When metamorphism occurs, there is usually little alteration in the bulk chemical composition of the rocks involved, that is, with the exception of water and volatile constituents such as carbon dioxide, little material is lost or gained. This type of alteration is described as an isochemical change. In contrast, allochemical changes are brought about by metasomatic processes that introduce material into or remove it from the rocks they affect. Metasomatic changes are brought about by hot gases or solutions permeating through rocks.

Metamorphic reactions are influenced by the presence of fluids or gases in the pores of the rocks concerned. For instance, due to the low conductivity of rocks, pore fluids may act as a medium of heat transfer. Not only does water act as an agent of transfer in metamorphism, but it also acts as a catalyst in many chemical reactions. It is a constituent in many minerals in metamorphic rocks of low and medium grade. Grade refers to the range of temperature under which metamorphism occurred.

Two major types of metamorphism may be distinguished on the basis of geological setting. One type is of local extent, whereas the other extends over a large area. The first type refers to thermal or contact metamorphism, and the latter refers to regional metamorphism. Another type of metamorphism is dynamic metamorphism, which is brought about by increasing stress. However, some geologists have argued that this is not a metamorphic process since it brings about deformation rather than transformation.

Metamorphic Textures and Structures

Most deformed metamorphic rocks possess some kind of preferred orientation. Preferred orientations may be exhibited as mesoscopic linear or planar structures that allow the rocks to split more easily in one direction than in others. One of the most familiar examples is cleavage in slate; a similar type of structure in metamorphic rocks of higher grade is schistosity. Foliation comprises a segregation of particular minerals into inconstant bands or contiguous lenticles that exhibit a common parallel orientation.

Slaty cleavage is probably the most familiar type of preferred orientation and occurs in rocks of low metamorphic grade (see also Chapter 2). It is characteristic of slates and phyllites (Fig. 1.11). It is independent of bedding, which it commonly intersects at high angles; and it reflects a highly developed preferred orientation of minerals, particularly of those belonging to the mica family.

Strain-slip cleavage occurs in fine-grained metamorphic rocks, where it may maintain a regular, though not necessarily constant, orientation. This regularity suggests some simple relationship between the cleavage and movement under regionally homogeneous stress in the final phase of deformation.

Harker (1939) maintained that schistosity develops in a rock when it is subjected to increased temperatures and stress that involves its reconstitution, which is brought about by localized solution of mineral material and recrystallization. In all types of metamorphisms, the growth of new crystals takes place in an attempt to minimize stress. When recrystallization occurs under conditions that include shearing stress, a directional element is imparted to the newly

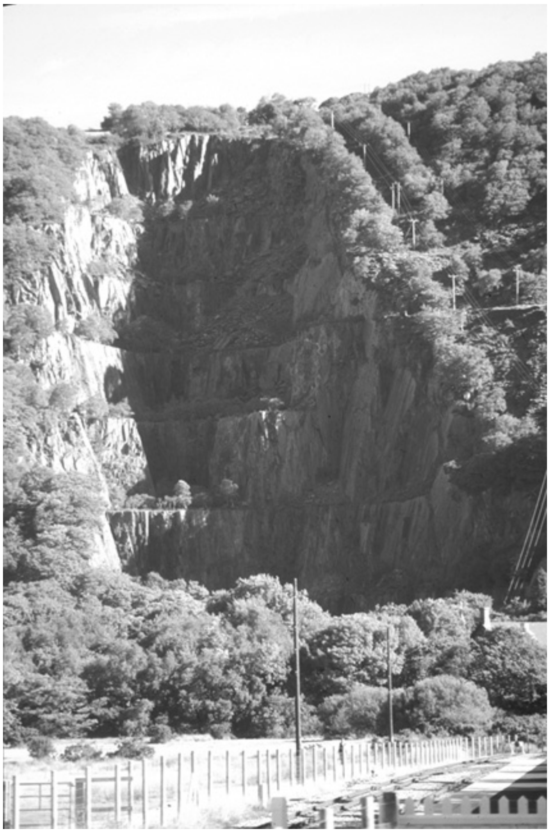
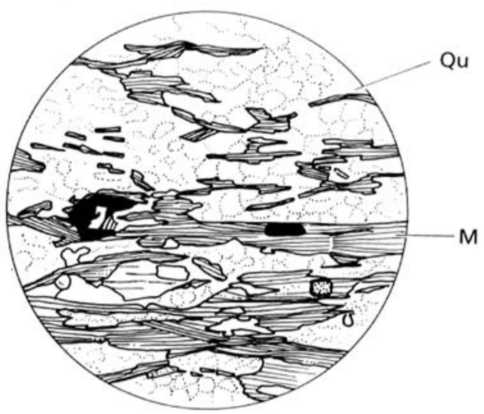


Figure 1.11

An old quarry in slate where extraction made use of the near-vertical cleavage, Nant Peris, North Wales.

formed rock. Minerals are arranged in parallel layers along the direction normal to the plane of shearing stress, giving the rock its schistose character (Fig. 1.12a and b). The most important minerals responsible for the development of schistosity are those that possess an acicular, flaky or tabular habit, the micas (e.g. muscovite) being the principal family involved. The more abundant flaky and tabular minerals are in such rocks, the more pronounced is the schistosity.

Foliation in a metamorphic rock is a very conspicuous feature, consisting of parallel bands or tabular lenticles formed of contrasting mineral assemblages such as quartz–feldspar and biotite–hornblende (Fig. 1.13a and b). It is characteristic of gneisses. This parallel orientation agrees with the direction of schistosity, if any is present in nearby rocks. Foliation, therefore, would seem to be related to the same system of stress and strain responsible for the development of schistosity. However, the influence of stress becomes less at higher temperatures



a

b

Figure 1.12

(a) Mica schist in which quartz and muscovite are segregated. Qu, quartz; M, muscovite (x 24). (b) Mica schist, northeast of Rhiconich, north of Scotland.

a

b

Figure 1.13

(a) Gneiss in which bands of quartz and feldspar are more or less separated from biotite and hornblende. Qu, quartz; F, feldspar; B, biotite; H, hornblende. (b) Banded and folded gneiss exposed north of Dombas, Norway.

and so schistosity tends to disappear in rocks of high-grade metamorphism. By contrast, foliation becomes a more significant feature. What is more, minerals of flaky habit are replaced in the higher grades of metamorphism by minerals such as garnet [$Fe_3Al_2(SiO_4)_3$], kyanite (Al_2SiO_5), sillimanite (Al_2SiO_5), diopside [$Ca,Mg(Si_2O_6)$] and orthoclase.

Thermal or Contact Metamorphism

Thermal metamorphism occurs around igneous intrusions so that the principal factor controlling these reactions is temperature. The rate at which chemical reactions take place during thermal metamorphism is exceedingly slow and depends on the rock types and temperatures involved. Equilibrium in metamorphic rocks, however, is attained more readily at higher grades because reaction proceeds more rapidly.

The encircling zone of metamorphic rocks around an intrusion is referred to as the contact aureole (Fig. 1.14). The size of an aureole depends on the size and temperature of the intrusion when emplaced, the quantity of hot gases and hydrothermal solutions that emanated from it,

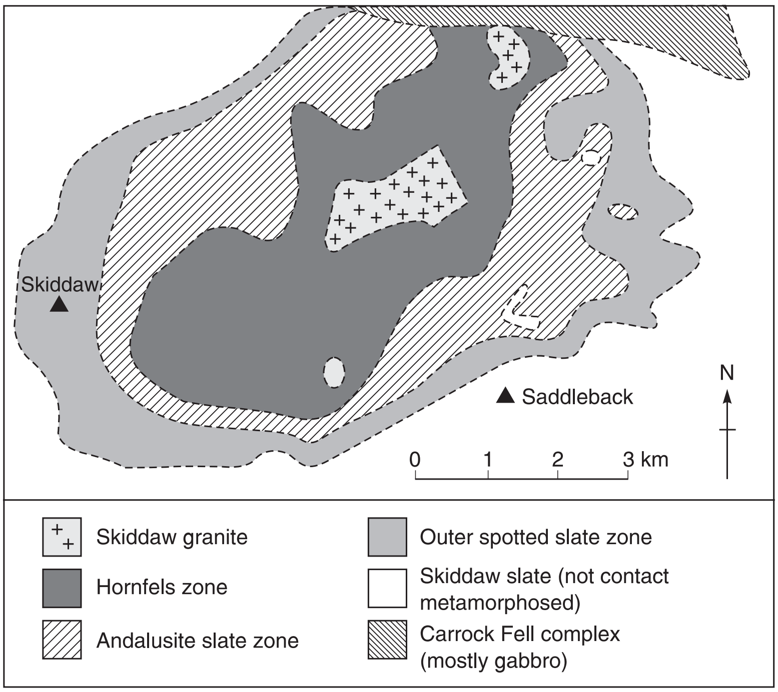


Figure 1.14

A sketch map of Skiddaw Granite and its contact aureole, Cumbria, England.

and the types of country rocks involved. Aureoles developed in argillaceous (or pelitic) sediments are more impressive than those found in arenaceous or calcareous rocks. This is because clay minerals, which account for a large proportion of the composition of argillaceous rocks, are more susceptible to temperature changes than quartz or calcite. Aureoles formed in igneous or previously metamorphosed terrains also are less significant than those developed in argillaceous sediments. Nevertheless, the capricious nature of thermal metamorphism must be emphasized, because the width of the aureole may vary even within one formation of the same rock type.

Within a contact aureole, there is usually a sequence of mineralogical changes from the country rocks to the intrusion, which have been brought about by the effects of a decreasing thermal gradient whose source was in the hot magma. Indeed, aureoles in argillaceous rocks may be concentrically zoned with respect to the intrusion. A frequently developed sequence varies inward from spotted slates to schists and then to hornfelses. Such an aureole is normally characterized mineralogically by chlorite [$(Ca,Fe,Mg)Al_2(Al_2Si_2)O_{10}(OH)_2$] and muscovite in the outer zone, biotite with or without andalusite (Al_2SiO_5) in the next zone, and biotite, cordierite [$(Mg,Fe)_2Al_3(AlSi_5)O_{18}$] and sillimanite ($Al_2SiO_5$) in the zone nearest the contact.

Hornfelses are characteristic products of high-grade thermal metamorphism. They are dark-coloured rocks with a fine-grained decussate, that is, interlocking texture, containing andalusite, cordierite, quartz, biotite, muscovite, microcline ($KAlSi_3O_8$) or orthoclase, and sodic plagioclase.

Aureoles formed in calcareous rocks frequently exhibit greater mineralogical variation and less regularity than do those in argillaceous rocks. Zoning, except on a small and localized scale, commonly is obscure. The width of the aureole and the mineral assemblage developed in the aureole appear to be related to the chemical composition and permeability of the parent calcareous beds. Marbles may be found in these aureoles, forming when limestone undergoes metamorphism.

The reactions that occur when arenaceous sediments are subjected to thermal metamorphism are usually less complicated than those that take place in their argillaceous or calcareous counterparts. For example, the metamorphism of a quartz arenite leads to the recrystallization of quartz to form quartzite with a mosaic texture; the higher the grade, the coarser the fabric. It is the impurities in sandstone that give rise to new minerals upon metamorphism. At high grades, foliation tends to develop and a gneissose rock is produced.

The acid and intermediate igneous rocks are resistant to thermal metamorphism; indeed, they are usually only affected at very high grades. For example, when granites are intruded by basic igneous masses, total recrystallization may be brought about in the immediate neighbourhood of the contact to produce a gneissose rock.

Basic igneous rocks undergo a number of changes when subjected to thermal metamorphism. They consist essentially of pyroxenes and plagioclase, and the first changes take place in the ferromagnesian minerals, that is, in the outermost region of an aureole the plagioclases are unaffected, thereby leaving the parental igneous texture intact. As the intrusion is approached, the rocks become completely recrystallized. At medium grade metamorphism, hornblende hornfelses are common. Nearest the contact, the high-grade rocks are typically represented by pyroxene hornfelses.

Regional Metamorphism

Metamorphic rocks extending over hundreds or even thousands of square kilometres are found exposed in the Pre-Cambrian shields, such as those that occur in Labrador and Fennoscandia, and in the eroded roots of fold mountains. As a consequence, the term regional has been applied to this type of metamorphism. Regional metamorphism involves both the processes of changing temperature and stress. The principal factor is temperature, which attains a maximum of around 800°C in regional metamorphism. Igneous intrusions are found within areas of regional metamorphism, but their influence is restricted. Regional metamorphism may be regarded as taking place when the confining pressures are in excess of 3 kilobars. What is more, temperatures and pressures conducive to regional metamorphism must have been maintained over millions of years. That temperatures rose and fell is indicated by the evidence of repeated cycles of metamorphism. These are not only demonstrated by mineralogical evidence but also by that of structures. For example, cleavage and schistosity are the results of deformation that is approximately synchronous with metamorphism but many rocks show evidence of more than one cleavage or schistosity that implies repeated deformation and metamorphism.

Regional metamorphism is a progressive process, that is, in any given terrain formed initially of rocks of similar composition, zones of increasing grade may be defined by different mineral assemblages. Each zone is defined by a significant mineral, and their mineralogical variation can be correlated with changing temperature–pressure conditions. The boundaries of each zone can therefore be regarded as isograds, that is, boundaries of equal metamorphic conditions.

Slates are the products of low-grade regional metamorphism of argillaceous or pelitic sediments. As the metamorphic grade increases, slates give way to phyllites in which somewhat larger crystals of chlorite and mica occur. Phyllites, in turn, give way to mica schists.

A variety of minerals such as garnet [$Fe_3Al_2(SiO_4)_3$], kyanite (Al_2SiO_5) and staurolite [$FeAl_4Si_2O_{10}(OH)_2$] may be present in these schists, indicating formation at increasing temperatures.

When sandstones are subjected to regional metamorphism, a quartzite develops that has a granoblastic (i.e. granular) texture. A micaceous sandstone or one in which there is an appreciable amount of argillaceous material, on metamorphism yields a quartz–mica schist. Metamorphism of arkoses and feldspathic sandstones leads to the recrystallization of feldspar and quartz so that granulites with a granoblastic texture are produced.

Relatively pure carbonate rocks when subjected to regional metamorphism simply recrystallize to form either calcite or dolomite marble with a granoblastic texture. Any silica present in a limestone tends to reform as quartz. The presence of micas in these rocks tends to give them a schistose appearance, schistose marbles or calc-schists being developed. Where mica is abundant, it forms lenses or continuous layers, giving the rock a foliated structure.

In regionally metamorphosed rocks derived from acid igneous parents, quartz and white mica are important components, muscovite–quartz schist being a typical product of the lower grades. In contrast, white mica is converted to potash feldspar at high grades. In the medium and high grades, quartzo–feldspathic gneisses and granulites are common. Some of the gneisses are strongly foliated.

Basic rocks are converted into greenschists by low-grade regional metamorphism, to amphibolites at medium grade, and to pyroxene granulites and eclogites at high grades.

Dynamic Metamorphism

Dynamic metamorphism is produced on a comparatively small scale and is usually highly localized; for example, its effects may be found in association with large faults or thrusts. On a larger scale, it is associated with folding, however, in the latter case, it may be difficult to distinguish between the processes and effects of dynamic metamorphism and those of low-grade regional metamorphism. What can be said is that at low temperatures, recrystallization is at a minimum and the texture of a rock is governed largely by the mechanical processes that have been operative. The processes of dynamic metamorphism include brecciation, cataclasis, granulation, mylonitization, pressure solution, partial melting and slight recrystallization.

Stress is the most important factor in dynamic metamorphism. When a body is subjected to stresses that exceed its limit of elasticity, it is permanently strained or deformed. If the stresses are equal in all directions, then the body simply undergoes a change in volume, whereas if they are directional, its shape is changed.

Brecciation is the process by which a rock is fractured, the angular fragments produced being of varying size. It is commonly associated with faulting and thrusting. The fragments of a crush breccia may themselves be fractured, and the mineral components may exhibit permanent strain phenomena. If pieces are rotated during the process of fragmentation, they are eventually rounded and embedded in the worn-down powdered material. The resultant rock is referred to as a crush conglomerate.

Mylonites are produced by the pulverization of rocks, which not only involves extreme shearing stress but also considerable confining pressure. Mylonitization is therefore associated with major faults. Mylonites are composed of strained porphyroblasts (metamorphic equivalent of phenocrysts) set in an abundant matrix of fine-grained or cryptocrystalline material. Quartzes in the groundmass are frequently elongated. Those mylonites that have suffered great stress lack porphyroblasts, having a laminated structure with a fine granular texture. The individual laminae are generally distinguishable because of their different colour. Protomylonite is transitional between micro-crush breccia and mylonite, while ultramylonite is a banded or structureless rock in which the material has been reduced to powder size.

In the most extreme cases of dynamic metamorphism, the resultant crushed material may be fused to produce a vitrified rock referred to as a pseudotachylite. It usually occurs as very small discontinuous lenticular bodies or branching veins in quartzite, amphibolite and gneiss. Quartz and feldspar fragments are usually found in a dark-coloured glassy base.

Metasomatism

Metasomatic activity involves the introduction of material into, as well as removal from, a rock mass by a hot gaseous or an aqueous medium, the resultant chemical reactions leading to mineral replacement. Thus, two types of metasomatism can be distinguished, namely, pneumatolytic (brought about by hot gases) and hydrothermal (brought about by hot solutions). Replacement occurs as a result of atomic or molecular substitution, so that there usually is little change in rock texture. The composition of the transporting medium is changing continuously because of material being dissolved out of and emplaced into the rocks that are affected.

The gases and hot solutions involved emanate from an igneous source, and the effects of metasomatism are often particularly notable about an intrusion of granitic character. Indeed, there is a greater concentration of volatiles in acid than in basic magmas.

Both gases and solutions make use of any structural weaknesses, such as faults, fissures or joint planes, in the rocks they invade. Because these provide easier paths for escape, metasomatic activity is concentrated along them. They also travel through the pore spaces in rocks, the rate of infiltration being affected by the porosity, the shape of the pores and the temperature–pressure gradients. Metasomatic action, especially when it is concentrated along fissure zones and veins, may bring about severe alteration of certain minerals. For instance, feldspars in granite or gneiss may be highly kaolinized as a result of metasomatism, and limestone may be reduced to a weakly bonded granular aggregate.

Sedimentary Rocks

The sedimentary rocks form an outer skin on the Earth's crust, covering three-quarters of the continental areas and most of the sea floor. They vary in thickness up to 10 km. Nevertheless, they only comprise about 5% of the crust.

Most sedimentary rocks are of secondary origin, in that they consist of detrital material derived by the breakdown of pre-existing rocks. Indeed, it has been variously estimated that shales and sandstones, both of mechanical derivation, account for between 75 and 95% of all sedimentary rocks. However, certain sedimentary rocks are the products of chemical or biochemical precipitation whereas others are of organic origin. Thus, the sedimentary rocks can be divided into two principal groups, namely, the clastic (detrital) or exogenetic, and the non-clastic or endogenetic types. Nevertheless, one factor that all sedimentary rocks have in common is that they are deposited, and this gives rise to their most noteworthy characteristic, that is, they are bedded or stratified.

As noted above, most sedimentary rocks are formed from the breakdown products of pre-existing rocks. Accordingly, the rate at which denudation takes place acts as a control on the rate of sedimentation, which in turn affects the character of a sediment. However, the rate of denudation is not only determined by the agents at work, that is, by weathering, or by river, marine, wind or ice action, but also by the nature of the surface. In other words, upland areas are more rapidly worn away than are lowlands. Indeed, denudation may be regarded as a cyclic process, in that it begins with or is furthered by the elevation of a land surface, and as this is gradually worn down, the rate of denudation slackens. Each cycle of erosion is accompanied by a cycle of sedimentation.

The particles of which most sedimentary rocks are composed have undergone varying amounts of transportation. The amount of transport together with the agent responsible, be it water, wind or ice, play an important role in determining the character of a sediment. For instance, transport over short distances usually means that the sediment is unsorted (the exception being beach sands), as does transportation by ice. With lengthier transport by water or wind, not only does the material become better sorted but it is further reduced in size.

The character of a sedimentary rock is also influenced by the type of environment in which it has been deposited, the presence of which is witnessed as ripple marks and cross bedding in sands that accumulate in shallow water.

The composition of a sedimentary rock depends partly on the composition of the parent material and the stability of its component minerals, and partly on the type of action to which the parent rock was subjected and the length of time it had to suffer such action. The least stable minerals tend to be those that are developed in environments very different from those experienced at the Earth's surface. In fact, quartz, and, to a much lesser extent, mica, are the only common detrital constituents of igneous and metamorphic rocks that are found in abundance in sediments. Most of the other minerals are ultimately broken down chemically to give rise to clay minerals. The more mature a sedimentary rock is, the more it approaches a stable end product, and very mature sediments are likely to have experienced more than one cycle of sedimentation.

The type of climatic regime in which a deposit accumulates and the rate at which this takes place also affect the stability and maturity of the resultant sedimentary product. For example, chemical decay is inhibited in arid regions so that less stable minerals are more likely to survive than in humid regions. However, even in humid regions, immature sediments may form when basins are rapidly filled with detritus derived from neighbouring mountains, the rapid burial affording protection against the attack of subaerial agents.

In order to turn unconsolidated sediment into solid rock, it must be lithified. Lithification involves two processes, consolidation and cementation. The amount of consolidation that takes place within a sediment depends, first, on its composition and texture and, second, on the pressures acting on it, notably that due to the weight of overburden. Consolidation of sediments deposited in water also involves dewatering, that is, the expulsion of connate water from the sediments. The porosity of a sediment is reduced as consolidation takes place, and, as the individual particles become more closely packed, they may even be deformed. Pressures developed during consolidation may lead to the differential solution of minerals and the authigenic growth of new ones.

Fine-grained sediments possess a higher porosity than do coarser types and, therefore, undergo a greater amount of consolidation. For instance, muds and clays may have original porosities ranging up to 80%, compared to 45 to 50% in sands and silts. Hence, if muds and clays could be completely consolidated (they never are), they would occupy only 20 to 45% of their original volume. The amount of consolidation that takes place in sands and silts varies from 15 to 25%.

Cementation involves the bonding together of sedimentary particles by the precipitation of material in the pore spaces. This reduces the porosity. The cementing material may be derived by partial intrastratal solution of grains or may be introduced into the pore spaces from an extraneous source by circulating waters. Conversely, cement may be removed from a sedimentary rock by leaching. The type of cement and, more importantly, the amount, affect the strength of a sedimentary rock. The type also influences its colour. For example, sandstones with siliceous or calcium carbonate cement are usually whitish grey, those with sideritic (iron carbonate) cement are buff coloured, whereas a red colour is indicative of hematitic (iron oxide) cement and brown of limonite (hydrated iron oxide). However, sedimentary rocks are frequently cemented by more than one material.

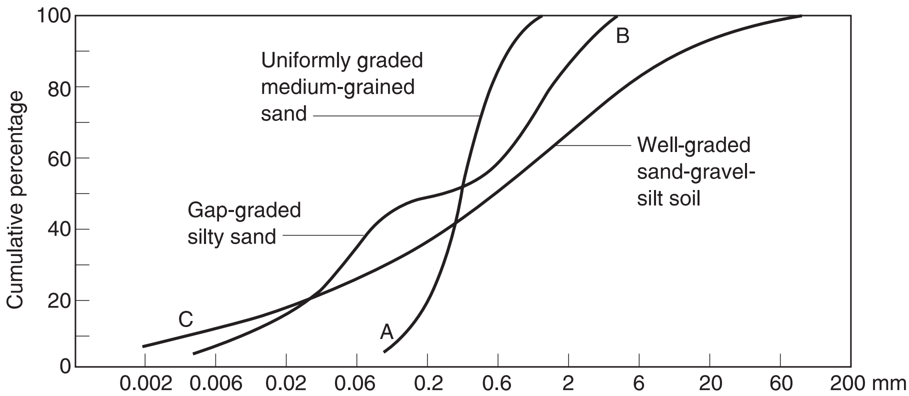
The matrix of a sedimentary rock refers to the fine material trapped within the pore spaces between the particles. It helps to bind the latter together.

The texture of a sedimentary rock refers to the size, shape and arrangement of its constituent particles. Size is a property that is not easy to assess accurately, for the grains and pebbles of which clastic sediments are composed are irregular, three-dimensional objects. Direct measurement can only be applied to large individual fragments where the length of the three principal axes can be recorded. But even this rarely affords a true picture of size. Estimation of volume by displacement may provide a better measure. Because of their smallness, the size of grains of sands and silts has to be measured indirectly by sieving and sedimentation techniques, respectively. If individual particles of clay have to be measured, this can be done with the aid of an electron microscope. If a rock is strongly indurated, its disaggregation is impossible without fracturing many of the grains. In such a case, a thin section of the rock is made and size analysis is carried out with the aid of a petrological microscope, mechanical stage and micrometer.

The results of a size analysis may be represented graphically by a frequency curve or histogram. More frequently, however, they are used to draw a cumulative curve. The latter may be drawn on semi-logarithmic paper (Fig. 1.15).

Various statistical parameters such as median and mean size, deviation, skewness and kurtosis can be calculated from data derived from cumulative curves. The median or mean size permits the determination of the grade of gravel, sand or silt, or their lithified equivalents. Deviation affords a measure of sorting. However, the latter can be quickly and simply estimated by visual examination of the curve in that the steeper it is, the more uniform the sorting of the sediment.

The size of the particles of a clastic sedimentary rock allows it to be placed in one of three groups that are termed rudaceous or psephitic, arenaceous or psammitic and argillaceous or pelitic. Reference to size scales is made in Chapter 5, where a description of mixed aggregates also is provided.



Clay	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	Cobbles Boulders
	Silt			Sand			Gravel			

Figure 1.15

Grading curves.

Shape is probably the most fundamental property of any particle, but, unfortunately it is one of the most difficult to quantify. Shape is frequently assessed in terms of roundness and sphericity, which may be estimated visually by comparison with standard images (Fig. 1.16). However, because the latter is a subjective assessment, the values obtained suffer accordingly.

A sedimentary rock is an aggregate of particles, and some of its characteristics depend on the position of these particles in space. The degree of grain orientation within a rock varies between perfect preferred orientation, in which all the long axes run in the same direction, and perfect random orientation, where the long axes point in all directions. The latter is found only infrequently as most aggregates possess some degree of grain orientation.

The arrangement of particles in a sedimentary rock involves the concept of packing, which refers to the spatial density of the particles in an aggregate. Packing has been defined as the mutual spatial relationship among the grains. It includes grain-to-grain contacts and the shape of the contact. The latter involves the closeness or spread of particles, that is, how much space in a given area is occupied by grains. Packing is an important property of sedimentary rocks, for it is related to their degree of consolidation, density, porosity and strength.

Bedding and Sedimentary Structures

Sedimentary rocks are characterized by their stratification, and bedding planes are frequently the dominant discontinuity in sedimentary rock masses (Fig. 1.17). As such, their spacing and

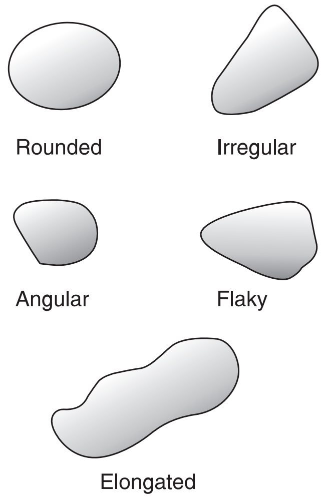


Figure 1.16

Images for estimating shape.

character (are they irregular, waved or straight, tight or open, rough or smooth?) are of particular importance to the engineer. Several spacing classifications have been advanced (see Chapter 2).

An individual bed may be regarded as a thickness of sediment of the same composition that was deposited under the same conditions. Lamination, on the other hand, refers to a bed of sedimentary rock that exhibits thin layers or laminae, usually a few millimetres in thickness. The laminae may be the result of minor fluctuations in the velocity of the transporting medium or the supply of material, both of which produce alternating thin layers of slightly different grain size. Generally, however, lamination is associated with the presence of thin layers of platy minerals, notably micas. These have a marked preferred orientation, usually parallel to the bedding planes, and are responsible for the fissility of the rock. The surfaces of these laminae are usually smooth and straight. Although lamination is most characteristic of shales, it also may be present in siltstones and sandstones, and occasionally in some limestones.

Cross or current bedding is a depositional feature that occurs in sediments of fluvial, littoral, marine and aeolian origin, and is found most notably in sandstones (Fig. 1.18). In wind-blown sediments, it generally is referred to as dune bedding. Cross bedding is confined within an individual sedimentation unit and consists of cross laminae inclined to the true bedding planes. The original dip of these cross laminae is frequently between 20 and 30°. The size of the sedimentation unit in which they occur varies enormously. For example, in micro-cross-bedding, it measures only a few millimetres, whereas in dune bedding, the unit may exceed 100 m.

Although graded bedding occurs in several different types of sedimentary rock, it is characteristic of greywacke. As the name suggests, the sedimentation unit exhibits a grading from coarser grain size at the bottom to finer at

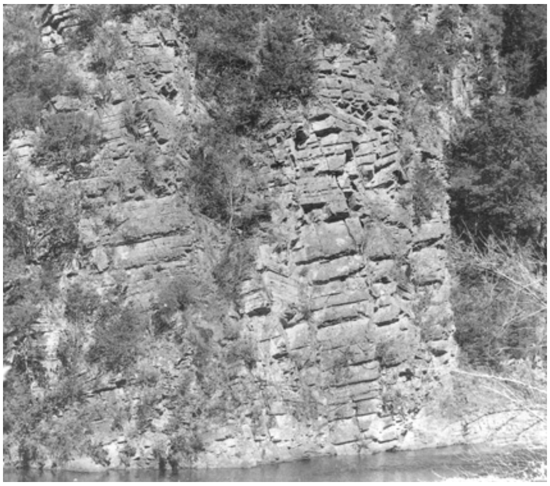


Figure 1.17

Bedding in sandstone, northwest of Nelson, South Island, New Zealand.

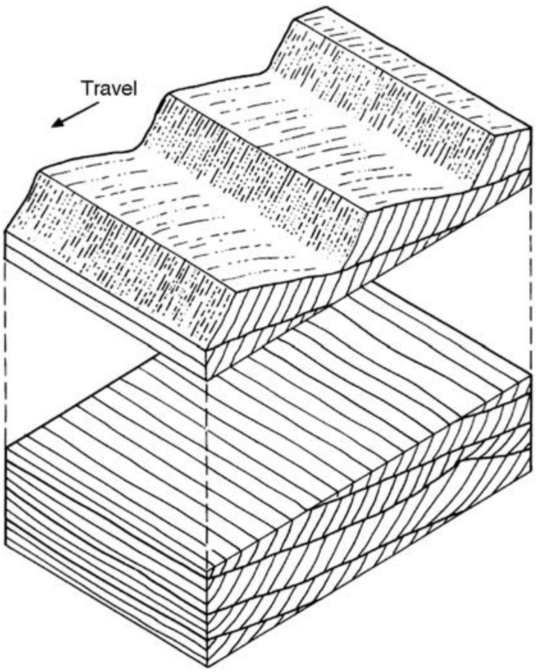


Figure 1.18

Diagram illustrating cross bedding.

the top. Individual graded beds range in thickness from a few millimetres to several metres. Usually, the thicker the bed, the coarser it is overall.

Sedimentary Rock Types

Gravel is an unconsolidated accumulation of rounded fragments, the lower size limit of which is 2 mm. The term rubble has been used to describe those deposits that contain angular fragments. The composition of a gravel deposit reflects not only the source rocks of the area from which it was derived but also is influenced by the agents responsible for its formation and the climatic regime in which it was (or is being) deposited. The latter two factors have a varying tendency to reduce the proportion of unstable material present. Relief also influences the nature of a gravel deposit. For example, gravel production under low relief is small, and the pebbles tend to be inert residues such as vein quartz, quartzite, chert and flint. Conversely, high relief and the accompanying rapid erosion yield coarse, immature gravels.

When gravel and larger-size material become indurated, they form conglomerate; when rubble is indurated, it is termed a breccia (Fig. 1.19). Those conglomerates in which the fragments are in contact and so make up a framework are referred to as orthoconglomerates. By contrast, those deposits in which the larger fragments are separated by matrix are referred to as paraconglomerates.

Sands consist of a loose mixture of mineral grains and rock fragments. Generally, they tend to be dominated by a few minerals, the chief of which is frequently quartz. Usually, the grains show some degree of orientation, presumably related to the direction of movement of the transporting medium.

The process by which sand is turned into sandstone is partly mechanical, involving grain fracturing, bending and deformation. However, chemical activity is much more important. The latter includes decomposition and solution of grains, precipitation of material from pore fluids and intergranular reactions. Silica (SiO_2) is the commonest cementing agent in sandstones, particularly older sandstones. Various carbonate cements, especially calcite ($CaCO_3$), are also common cementing materials. Ferruginous and gypsiferous cements also are found in sandstones. Cement, notably the carbonate types, may be removed in solution by percolating pore fluids. This brings about varying degrees of decementation.

Quartz, feldspar and rock fragments are the principal detrital components of which sandstones are composed, and consequently they have been used to define the major classes of sandstone. Pettijohn et al. (1972) also used the type of matrix in their classification.

Figure 1.19

A conglomerate in the Old Red Sandstone, north of Belfast, Northern Ireland.

In other words, those sandstones with more than 15% matrix were termed wackes. The chief type of wacke is greywacke, which can be subdivided into lithic and feldspathic varieties. Those sandstones with less than 15% matrix were divided into three families. The orthoquartzites or quartz arenites contain 95% or more of quartz; 25% or more of the detrital material in arkoses consists of feldspar; and in lithic sandstones, 25% or more of the detrital material consists of rock fragments (Fig. 1.20).

Silts are clastic sediments derived from pre-existing rocks, chiefly by mechanical breakdown processes. They are composed mainly of fine quartz material. Silts may occur in residual soils, but they are not important in such instances. However, silts are commonly found in alluvial, lacustrine, fluvio-glacial and marine deposits. These silts tend to interdigitate with deposits of sand and clay. Silts are also present with sands and clays in estuarine and deltaic sediments. Lacustrine silts are often banded. Marine silts may also be banded. Wind-blown silts are generally uniformly sorted.

Siltstones may be massive or laminated, the individual laminae being picked out by mica and/or carbonaceous material. Micro-cross-bedding is frequently developed and the laminations may be convoluted in some siltstones. Siltstones have high quartz content with predominantly

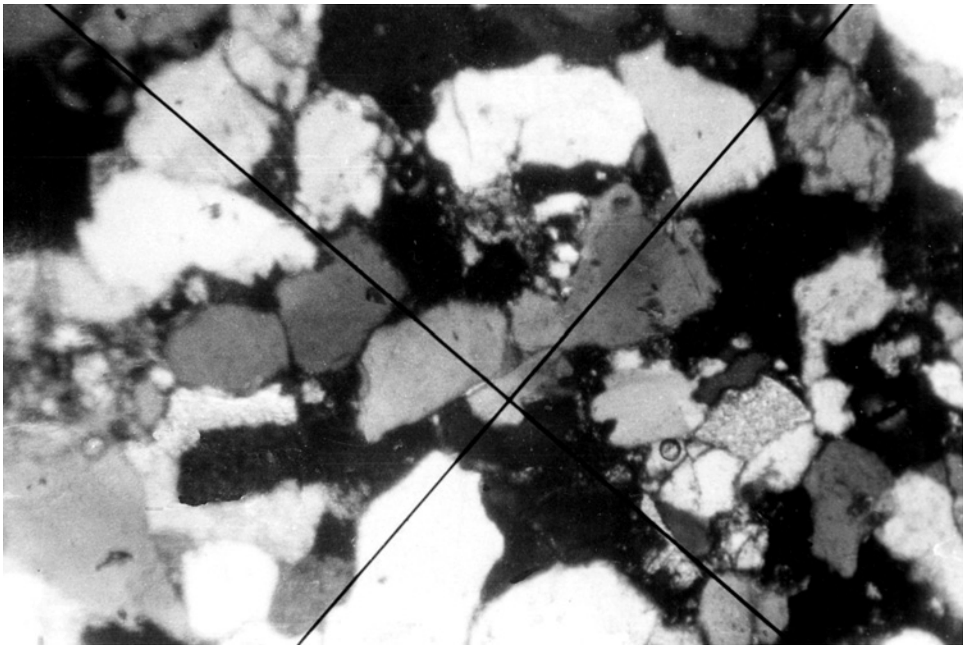


Figure 1.20

Thin section of Fell Sandstone (a quartz arenite), Lower Carboniferous, Northumberland, England.

siliceous cement. Frequently, siltstones are interbedded with shales or fine-grained sandstones, the siltstones occurring as thin ribs.

Loess is a wind-blown deposit that is mainly of silt size and consists mostly of quartz particles, with lesser amounts of feldspar and clay minerals. It is characterized by a lack of stratification and uniform sorting, and occurs as blanket deposits in western Europe, the United States, Russia and China (Fig. 1.21). Deposits of loess are of Pleistocene age and, because they show a close resemblance to fine-grained glacial debris, their origin has customarily been assigned a glacial association. For instance, in the case of those regions mentioned, it is presumed that winds blowing from the arid interiors of the northern continents during glacial times picked up fine glacial outwash material and carried it for hundreds or thousands of kilometres before deposition took place. Deposition is assumed to have occurred over steppe lands, and the grasses left behind fossil root holes, which typify loess. These account for its crude columnar structure. The lengthy transport explains the uniform sorting of loess.

Deposits of clay are composed principally of fine quartz and clay minerals. The latter represent the commonest breakdown products of most of the chief rock-forming silicate minerals.

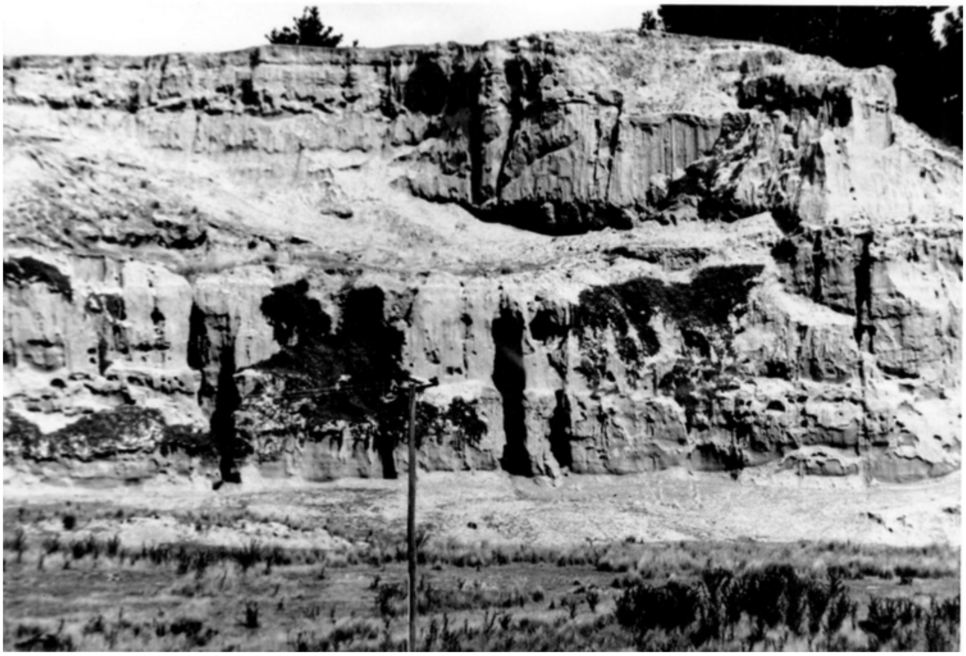


Figure 1.21

A deposit of loess near Kansas City, United States.

The clay minerals are all hydrated aluminium silicates and possess a flaky habit, that is, they are phyllosilicates. The three major families of clay minerals are the kandites (kaolinite), illites (illite) and smectites (montmorillonite).

Kaolinite [$Al_4Si_4O_{10}(OH)_8$] is formed by the alteration of feldspars, feldspathoids and other aluminium silicates due to hydrothermal action. Weathering under acidic conditions is also responsible for kaolinization. Kaolinite is the chief clay mineral in most residual and transported clays, is important in shales, and is found in variable amounts in fireclays, laterites and soils. It is the most important clay mineral in china clays and ball clays. Deposits of china clay (kaolin) are associated with acid igneous rocks such as granites, granodiorites and tonalites, and with gneisses and granulites.

Illite [$K_{2\text{-}3}Al_8(Al_{2\text{-}3},Si_{13\text{-}14})O_{40}(OH)_8$] is of common occurrence in clays and shales, and is found in variable amounts in tills and loess, but is less common in soils. It develops as an alteration product of feldspars, micas or ferromagnesian silicates upon weathering or may form from other clay minerals during diagenesis. Like kaolinite, illite also may be of hydrothermal origin. The development of illite, both under weathering and by hydrothermal processes, is favoured by an alkaline environment.

Montmorillonite $[(Mg,Al)_4(Al,Si)_8O_{20}(OH)_4.nH_2O]$ develops when basic igneous rocks in badly drained areas are subjected to weathering. The presence of magnesium is necessary for this mineral to form, if the rocks were well drained, then the magnesium would be carried away and kaolinite would develop. An alkaline environment favours the formation of montmorillonite. Montmorillonite occurs in soils and argillaceous sediments such as shales derived from basic igneous rocks. It is the principal constituent of bentonitic clays, which are formed by the weathering of basic volcanic ash, and of fuller's earth, which is also formed when basic igneous rocks are weathered. In addition, when basic igneous rocks are subjected to hydrothermal action, this may lead to the development of montmorillonite.

Residual clay deposits develop in place and are the products of weathering. In humid regions, residual clays tend to become enriched in hydroxides of ferric iron and aluminium, and impoverished in lime, magnesia and alkalies. Even silica is removed in hot humid regions, resulting in the formation of hydrated alumina or iron oxide, as in laterite.

The composition of transported clays varies because these materials consist mainly of abrasion products (usually silty particles) and transported residual clay material.

Shale is the commonest sedimentary rock and is characterized by its lamination. Sedimentary rock of similar size range and composition, but which is not laminated, is referred to as mudstone. In fact, there is no sharp distinction between shale and mudstone, one grading into the other. An increasing content of siliceous or calcareous material decreases the fissility of shale, whereas shales that have a high organic content are finely laminated. Laminae range from 0.05 to 1.0 mm in thickness, with most in the range of 0.1 to 0.4 mm. Clay minerals and quartz are the principal constituents of mudstones and shales. Feldspars often occur in the siltier shales. Shale may also contain appreciable quantities of carbonate, particularly calcite, and gypsum ($CaSO_4.2H_2O$). Indeed, calcareous shale frequently grades into shaly limestone. Carbonaceous black shales are rich in organic matter, contain a varying amount of pyrite (FeS_2), and are finely laminated.

The term limestone is applied to those rocks in which the carbonate fraction exceeds 50%, over half of which is calcite or aragonite ($CaCO_3$). If the carbonate material is made up chiefly of dolomite ($CaCO_3.MgCO_3$), the rock is named dolostone (this rock generally is referred to as dolomite, but this term can be confused with that of the mineral of the same name). Limestones and dolostones constitute about 20 to 25% of the sedimentary rocks, according to Pettijohn (1975). This figure is much higher than some of the estimates provided by previous authors. Limestones are polygenetic. Some are of mechanical origin, representing carbonate detritus that has been transported and deposited. Others represent chemical or biochemical precipitates that have formed in place. Allochthonous or transported limestone has a fabric similar to that of sandstone and also may display current structures such as cross bedding and ripple marks. By contrast, carbonate rocks that have formed in situ, that is, autochthonous types, show no evidence of sorting or current action and at best possess a poorly developed stratification. Exceptionally, some autochthonous limestones show growth bedding, the most striking of which is stromatolitic bedding, as seen in algal limestones.

Lithification of carbonate sediments often is initiated as cementation at points of intergranular contact rather than as consolidation. In fact, carbonate muds consolidate very little because of this early cementation. The rigidity of the weakest carbonate rocks, such as chalk, may be attributed to the mechanical interlocking of grains with little or no cement. Although cementation may take place more or less at the same time as deposition, cemented and uncemented assemblages may be found within short horizontal distances. Indeed, a recently cemented carbonate layer may overlie uncemented material. Because cementation occurs concurrently with or soon after deposition, carbonate sediments can support high overburden pressures before consolidation takes place. Hence, high values of porosity may be retained to considerable depths of burial. Eventually, however, the porosity is reduced by post-depositional changes that bring about recrystallization. Thus, a crystalline limestone is formed in this manner.

Folk (1973) distinguished two types of dolostone. First, he recognized an extremely fine-grained crystalline dolomicrite (less than 20 microns grain diameter), and secondly, a more coarsely grained dolostone in which there was plentiful evidence of replacement. He regarded the first type as of primary origin and the second as being formed as a result of diagenetic replacement of calcite by dolomite in limestone. Primary dolostones tend to be thinly laminated and generally are unfossiliferous. They are commonly associated with evaporates and may contain either nodules or scattered crystals of gypsum or anhydrite ($CaSO_4$). In those dolostones formed by dolomitization, the original textures and structures may be obscured or may even have disappeared.

Evaporitic deposits are quantitatively unimportant as sediments. They are formed by precipitation from saline waters, the high salt content being brought about by evaporation from inland seas or lakes in arid areas. Salts can also be deposited from subsurface brines, brought to the surface of a playa or sabkha flat by capillary action (Fig. 1.22). Seawater contains approximately 3.5%, by weight, of dissolved salts, about 80% of which is sodium chloride. Experimental work has shown that when the original volume of seawater is reduced by evaporation to about half, a little iron oxide and some calcium carbonate are precipitated. Gypsum begins to form when the volume is reduced to about one-fifth of the original, rock salt begins to precipitate when about one-tenth of the volume remains, and, finally, when only 1.5% of the seawater is left, potash and magnesium salts start to crystallize. This order agrees in a general way with the sequences found in some evaporitic deposits, however, many exceptions are known. Many complex replacement sequences occur among evaporitic

Figure 1.22

Salt teepees on the Devil's Golf Course, a salina, Death Valley, California.

rocks, for example, carbonate rocks may be replaced by anhydrite and sulphate rocks by halite (NaCl).

Organic residues that accumulate as sediments are of two major types, namely, peaty material that when buried gives rise to coal, and sapropelic residues. Sapropel is silt rich in, or composed wholly of, organic compounds that collect at the bottom of still bodies of water. Such deposits may give rise to cannel or boghead coals. Sapropelic coals usually contain a significant amount of inorganic matter as opposed to humic coals in which the inorganic content is low. The former are generally not extensive and are not underlain by seat earths (i.e. fossil soils). Peat deposits accumulate in poorly drained environments in which the formation of humic acid gives rise to deoxygenated conditions. These inhibit the bacterial decay of organic matter. Peat accumulates wherever the deposition of plant debris exceeds the rate of its decomposition. A massive deposit of peat is required to produce a thick seam of coal; for example, a seam 1 m thick probably represents 15 m of peat.

Chert and flint are the two most common siliceous sediments of chemical origin. Chert is a dense rock composed of one or more forms of silica such as opal, chalcedony or microcrystalline quartz. Sponge spicules and radiolarian remains may be found in some cherts, and carbonate material may be scattered throughout impure varieties. Gradations occur from chert to sandstone with chert cement, although sandy cherts are not common. Chert may suffer varying degrees of devitrification. Chert may occur as thin beds or as nodules in carbonate host rocks. Both types are of polygenetic origin. In other words, chert may be a replacement product, as in siliceous limestone, for example, or it may represent a biochemical accumulate formed in a basin below the calcium carbonate compensation depth. In yet other cases, chert may be the product of an ephemeral silica-rich alkaline lake environment.

Some sediments may have a high content of iron. The iron carbonate, siderite ($FeCO_3$), often occurs interbedded with chert or mixed in varying proportions with clay, as in clay ironstones. Some iron-bearing formations are formed mainly of iron oxide, hematite (Fe_2O_3) being the most common mineral. Hematite-rich beds are generally oolitic. Limonite ($2Fe_2O_3.3H_2O$) occurs in oolitic form in some ironstones. Bog iron ore is chiefly an earthy mixture of ferric hydroxides. Siliceous iron ores include chamositic ironstones (chamosite, $Fe_3Al_2Si2O_{10}.3H_2O$), which are also typically oolitic. Glauconitic [glauconite, $K(Fe^3Al)_2(Si,Al)_4O_{10}(OH)_2$] sandstones and limestones may contain 20% or more FeO and Fe_2O_3. On rare occasions, bedded pyrite has been found in black shale.

Stratigraphy and Stratification

Stratigraphy is the branch of geology that deals with the study and interpretation of stratified rocks, and with the identification, description, sequence, both vertical and horizontal, mapping and correlation of stratigraphic rock units. As such, it begins with the discrimination and description of stratigraphical units such as formations. This is necessary so that the complexities present in every stratigraphical section may be simplified and organized.

Deposition involves the build-up of material on a given surface, either as a consequence of chemical or biological growth or, far more commonly, due to mechanically broken particles being laid down on such a surface. The changes that occur during deposition are responsible for stratification, that is, the layering that characterizes sedimentary rocks. A simple interruption of deposition ordinarily does not produce stratification. The most obvious change that gives rise to stratification is in the composition of the material being deposited. Even minor changes in the type of material may lead to distinct stratification, especially if they affect the colour of the rocks concerned. Changes in grain size may also cause notable layering, and changes in other textural characteristics may help distinguish one bed from another, as may variations in the degree of consolidation or cementation.

The extent and regularity of beds of sedimentary rocks vary within wide limits. This is because lateral persistence and regularity of stratification reflect the persistence and regularity of the agent responsible for deposition. For instance, sands may have been deposited in one area whereas muds were being deposited in a neighbouring area. What is more, a formation with a particular lithology, which is mappable as a stratigraphic unit, may not have been laid down at the same time wherever it occurs. The base of such a formation is described as diachronous (Fig. 1.23). Diachronism is brought about when a basin of deposition is advancing or retreating as, for example, in marine transgression or regression. In an expanding basin, the lowest sediments to accumulate are not as extensive as those succeeding. The latter are said to overlap the lowermost deposits. Conversely, if the basin of deposition is shrinking, the opposite situation arises in that succeeding beds are less extensive. This phenomenon is termed offlap.

Agents confined to channels or responsible for deposition over limited areas produce irregular strata that are not persistent. By contrast, strata that are very persistent are produced by agents operating over wide areas. In addition, folding and faulting of strata, along with subsequent erosion, give rise to discontinuous outcrops.

Since sediments are deposited, it follows that the topmost layer in any succession of strata is the youngest. Also, any particular stratum in a sequence can be dated by its position in the sequence relative to other strata. This is the Law of Superposition. This principle applies to all sedimentary rocks except, of course, those that have been overturned by folding or where older strata have been thrust over younger rocks. Where strata are overfolded, the stratigraphical succession is inverted. When fossils are present in the beds concerned, their

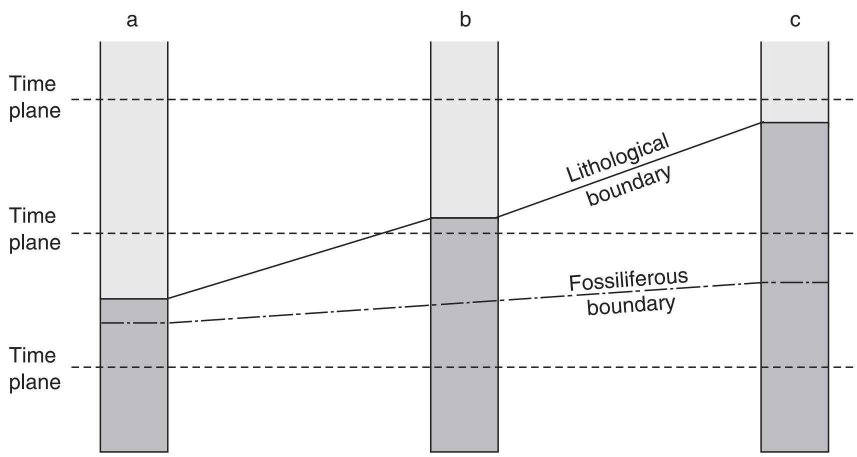


Figure 1.23

Diachronism of a lithological boundary and the migration time of a fossil assemblage. The fossiliferous horizon may be regarded as a time plane if the localities (a), (b) and (c) are not far distant. As a rule, time planes cannot be identified.

correct way up can be discerned. However, if fossil evidence is lacking, the correct way up of the succession may be determined from evidence provided by the presence of "way-up" structures such as graded bedding, cross bedding and ripple marks (Shrock, 1948).

Unconformities

An unconformity represents a break in the stratigraphical record and occurs when changes in the palaeogeographical conditions lead to a cessation of deposition for a period of time. Such a break may correspond to a relatively short interval of geological time or a very long one. An unconformity normally means that uplift and erosion have taken place, resulting in some previously formed strata being removed. The beds above and below the surface of unconformity are described as unconformable.

The structural relationship between unconformable units allows four types of unconformity to be distinguished. In Figure 1.24a, stratified rocks rest upon igneous or metamorphic rocks. This type of feature has been referred to as a nonconformity (it also has been called a heterolithic unconformity). An angular unconformity is shown in Figure 1.24b, where an angular discordance separates the two units of stratified rocks. In an angular unconformity,

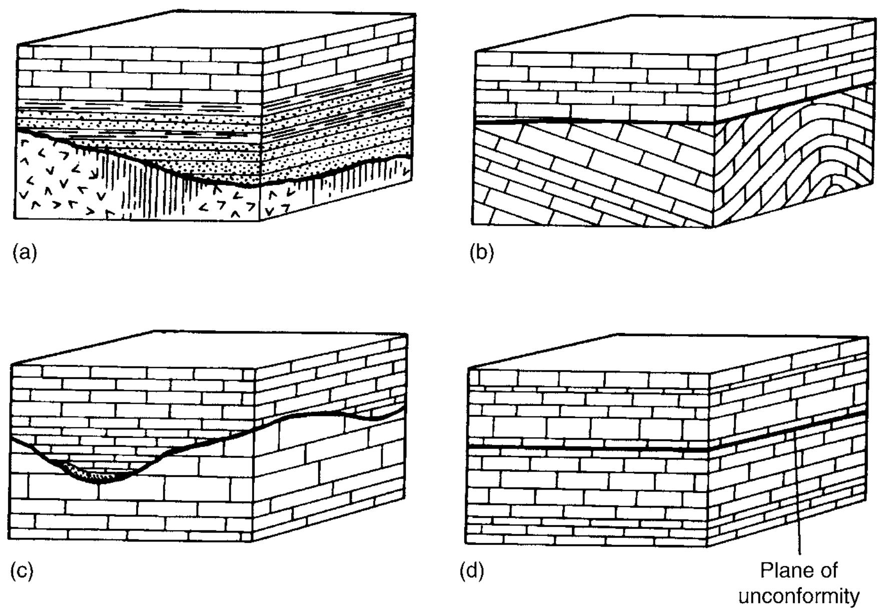


Figure 1.24

Types of unconformities: (a) nonconformity or heterolithic unconformity, (b) angular unconformity, (c) disconformity and (d) paraconformity.

the lowest bed in the upper sequence of strata usually rests on beds of differing ages. This is referred to as overstep. In a disconformity, as illustrated in Figure 1.24c, the beds lie parallel both above and below the unconformable surface, but the contact between the two units concerned is an uneven surface of erosion. When deposition is interrupted for a significant period but there is no apparent erosion of sediments or tilting or folding, then subsequently formed beds are deposited parallel to those already existing. In such a case, the interruption in sedimentation may be demonstrable only by the incompleteness of the fossil sequence. This type of unconformity has been termed a paraconformity (Fig. 1.24d).

One of the most satisfactory criteria for the recognition of unconformities is evidence of an erosion surface between two formations. Such evidence may take the form of pronounced irregularities in the surface of the unconformity. Evidence also may take the form of weathered strata beneath the unconformity, weathering having occurred prior to the deposition of the strata above. Fossil soils provide a good example. The abrupt truncation of bedding planes, folds, faults, dykes, joints, etc., in the beds below the unconformity is characteristic of an unconformity (Fig. 1.25), although large-scale thrusts will give rise to a similar structural arrangement. Post-unconformity sediments often commence with a conglomeratic deposit. The pebbles in the conglomerate may be derived from the older rocks below the unconformity.

Figure 1.25

Angular unconformity between highly folded Horton Flags, Silurian, below and almost horizontal Lower Carboniferous Limestone above, Helwith Bridge, North Yorkshire, England.

Rock and Time Units

Stratigraphy distinguishes rock units and time units. A rock unit, such as a stratum or a formation, possesses a variety of physical characteristics that enable it to be recognized as such, and, hence, measured, described, mapped and analysed. A rock unit is sometimes termed a lithostratigraphical unit.

A particular rock unit required a certain interval of time for it to form. Hence, stratigraphy not only deals with strata but also with age, and the relationship between strata and age. Accordingly, time units and time-rock units have been recognized. Time units are simply intervals of time, the largest of which are eons, although this term tends to be used infrequently. There are two eons, representing Pre-Cambrian time and Phanerozoic time. Eons are divided into eras, and eras into periods (Table 1.1). Periods are, in turn, divided into epochs and epochs into ages. Time units and time-rock units are directly comparable, that is, there is a corresponding time-rock unit for each time unit. For example, the time-rock unit corresponding to a period is a system. Indeed, the time allotted to a time unit is determined from the rocks of the corresponding time-rock unit.

A time-rock unit has been defined as a succession of strata bounded by theoretically uniform time planes, regardless of the local lithology of the unit. Fossil evidence usually provides the basis for the establishment of time planes. Ideal time-rock units would be bounded by completely independent time planes, however, practically the establishment of time-rock units depend on whatever evidence is available.

Geological systems are time-rock units that are based on stratigraphical successions present in certain historically important areas. In other words, in their type localities, the major time-rock units are also rock units. The boundaries of major time-rock units are generally important structural or faunal breaks or are placed at highly conspicuous changes in lithology. Major unconformities are frequently chosen as boundaries. Away from their type areas, major time-rock units may not be so distinctive or easily separated. In fact, although systems are regarded as of global application, there are large regions where the recognition of some of the systems has not proved satisfactory.

Correlation

The process by which the time relationships between strata in different areas are established is referred to as correlation. Correlation is, therefore, the demonstration of equivalency of stratigraphical units. Palaeontological and lithological evidence are the two principal criteria used in correlation. The principle of physical continuity may be of some use in local correlation. In other words, it can be assumed that a given bed, or bedding plane, is roughly

Table 1.1. The geological timescale

Eras	Periods and systems	Derivation of names	Duration of period (Ma)	Total from beginning (Ma)
Cainozoic	Quaternary			
	Recent *or* Holocene*	*Holos* = complete whole		
	Glacial *or* Pleistocene*	*Pleiston* = most	2 or 3	2–3
	Tertiary			
	Pliocene*	*Pleion* = more	9 or 10	12
	Miocene*	*Meion* = less (i.e. less than in Pliocene)	13	25
	Oligocene*	*Oligos* = few	15	40
	Eocene*	*Eos* = dawn	20	60
	Paleocene*	*Palaios* = old	10	70
	The preceding comparative terms refer to the proportions of modern marine shells occuring as fossils.			
Mesozoic	Cretaceous	*Creta* = chalk	65	135
	Jurassic	*Jura* Mountains	45	180
	Triassic	*Threefold* division in Germany	45	225
	(New Red Sandstone = desert sandstones of the Triassic period and part of the Permian)			
Palaeozoic	Permian	*Permia*, ancient kingdom between the Urals and the Volga	45	270
	Carboniferous	*Coal* (carbon)-bearing	80	350
	Devonian	*Devon* (marine sediments)		
	(Old Red Sandstone = land sediments of the Devonian period)		50	400
	Silurian	*Silures*, Celtic tribe of Welsh borders	40	440
	Ordovician	*Ordovices*, Celtic tribe of North Wales	60	500
	Cambrian	*Cambria*, Roman name for Wales	100	600
Pre-Cambrian Era				
Origin of Earth				5000

*Frequently regarded as epochs or stages, “cene” from *kainos* = recent.

contemporaneous throughout an outcrop of bedded rocks. Tracing of bedding planes laterally, however, may be limited since individual beds or bedding planes die out, are interrupted by faults, are missing in places due to removal by erosion, are concealed by overburden, or merge with others laterally. Consequently, outcrops may not be good enough to permit an individual bed to be traced laterally over an appreciable distance. A more practicable procedure is to trace a member of a formation. However, this also can prove misleading if the beds are diachronous.

Where outcrops are discontinuous, physical correlation depends on lithological similarity, that is, on matching rock types across the breaks in hope of identifying the beds involved. The lithological characters used to make comparison in such situations include gross lithology, subtle distinctions within one rock type such as a distinctive heavy mineral suite, notable microscopic features or distinctive key beds. The greater the number of different, and especially unusual, characters that can be linked, the better is the chance of reliable correlation. Even so, such factors must be applied with caution and, wherever possible, such correlation should be verified by the use of fossils.

If correlation can be made from one bed in a particular outcrop to one in another, it can be assumed that the beds immediately above and below also are correlative, provided, of course, that there was no significant break in deposition at either exposure. Better still, if two beds can be correlated between two local exposures, then the intervening beds are presumably correlative, even if the character of the intervening rocks is different in the two outcrops. This again depends on there being no important break in deposition at either of the locations.

At the end of the eighteenth century, William Smith formulated the Law of Faunal Succession, which states that strata of different ages are characterized by different fossils or suites of fossils. Smith demonstrated that each formation could be identified by its distinctive suite of fossils without the need of lateral tracing. In this way, he developed the use of guide fossils as a method of recognizing rocks of equivalent age. The recognition of strata by their fossil content depends on the fact that species and genera become extinct, with new ones replacing them.

As far as correlation is concerned, good fossils should have a wide geographical distribution and a limited stratigraphical range. In general, groups of organisms that possessed complicated structures provide better guides for correlation than those that were simple. The usefulness of fossils is enhanced if the group concerned evolved rapidly, for where morphological changes took place rapidly, individual species existed for a relatively short duration. These fossils provide a more accurate means of subdividing the geological column and, therefore, provide more precise correlation. Groups that were able to swim or float prove especially useful since they ranged widely and were little restricted in distribution by the conditions on the sea floor.

The principal way in which fossils are used in correlation is based on the recognition of characteristic species in strata of a particular age. This method can be applied in two ways. First, index fossils can be established, which in turn allows a particular bed to be identified in terms of geological time. Second, fossils may be used to distinguish zones. A zone may be defined as that strata that was laid down during a particular interval of time when a given fauna or flora existed. In some cases, zones have been based on the complete fauna present, whereas in other instances they have been based on members of a particular phylum or class. Nonetheless, a zone is a division of time given in terms of rocks deposited. Although a faunal or floral zone is defined by reference to an assemblage of fossils, it is usually named after some characteristic species, and this fossil is known as the zone fossil. Normally, a faunal or floral zone is identifiable because certain species existed together for some time. It is assumed that these species have time ranges that are overlapping and that their time ranges are similar in different areas.

Geological Structures

The two most important features that are produced when strata are deformed by earth movements are folds and faults, that is, the rocks are buckled or fractured, respectively. A fold is produced when a more or less planar surface is deformed to give a waved surface. On the other hand, a fault represents a surface of discontinuity along which the strata on either side have been displaced relative to each other.

Folds

Anatomy of Folds

There are two important directions associated with inclined strata, namely, dip and strike. True dip gives the maximum angle at which a bed of rock is inclined and should always be distinguished from apparent dip (Fig. 2.1). The latter is a dip of lesser magnitude whose direction can run anywhere between that of true dip and strike. Strike is the trend of inclined strata and is orientated at right angles to the true dip, it has no inclination (Fig. 2.1).

Folds are wave-like in shape and vary enormously in size. Simple folds are divided into two types, that is, anticlines and synclines (Fig. 2.2a and b). In the former, the beds are convex upwards, whereas in the latter, they are concave upwards. The crestal line of an anticline is the line that joins the highest parts of the fold, whereas the trough line runs through the lowest parts of a syncline (Fig. 2.2a). The amplitude of a fold is defined as the vertical difference between the crest and the trough, whereas the length of a fold is the horizontal distance from crest to crest or trough to trough. The hinge of a fold is the line along which the greatest curvature exists and can be either straight or curved. However, the axial line is another term that has been used to describe the hinge line. The limb of a fold occurs between the hinges, all folds having two limbs. The axial plane of a fold is commonly regarded as the plane that bisects the fold and passes through the hinge line.

The inter-limb angle, which is the angle measured between the two projected planes from the limbs of the fold, can be used to assess the degree of closure of a fold. Five degrees of closure can be distinguished based on the inter-limb angle. Gentle folds are those with an

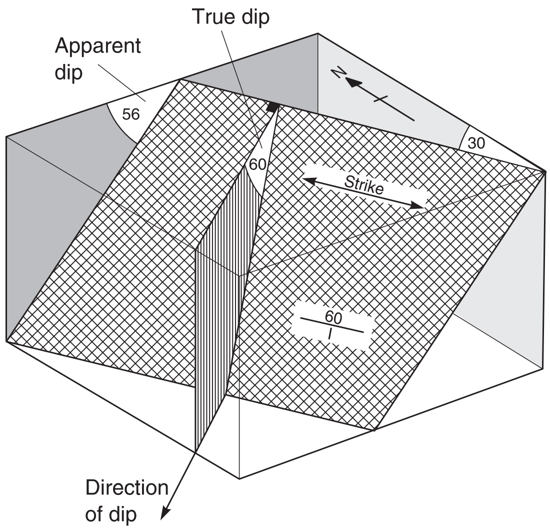


Figure 2.1

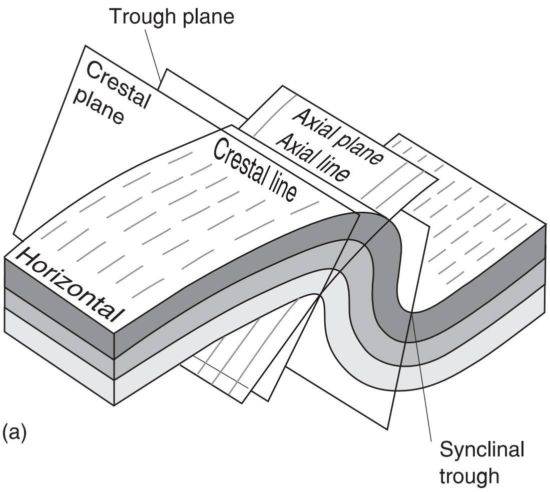
Illustration of dip and strike: orientation of cross-hatched plane can be expressed as strike 330°, dip 60°.

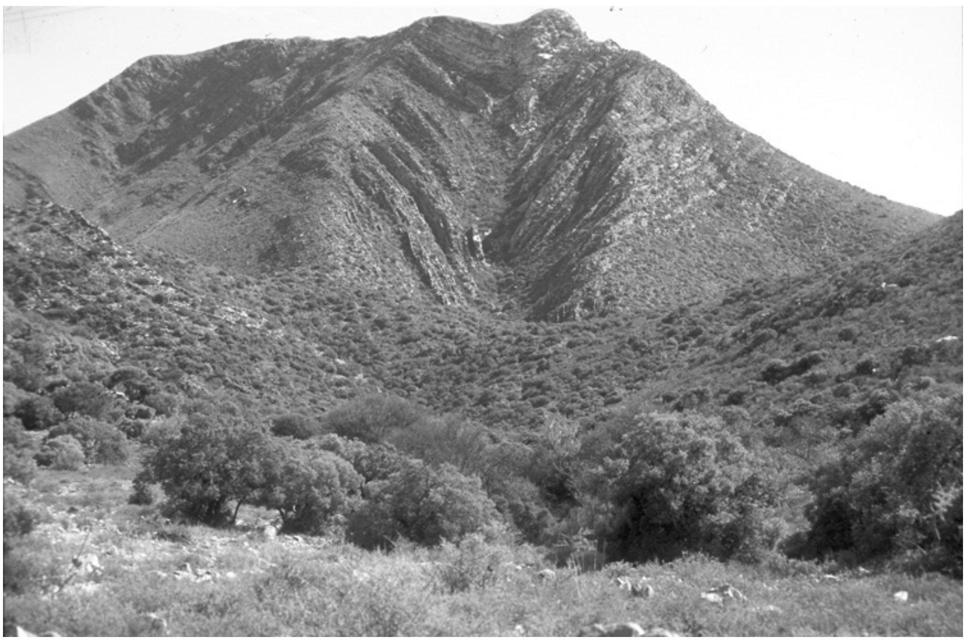
inter-limb angle greater than 120°; in open folds, the inter-limb angle is between 120 and 70°; in close folds, it is between 70 and 30°; tight folds are those with an inter-limb angle of less than 30° and, finally, in isoclinal folds, the limbs are parallel and so the inter-limb angle is zero.

Folds are of limited extent and, when one fades out, the attitude of its axial line changes, that is, it dips away from the horizontal. This is referred to as the plunge or pitch of the fold (Fig. 2.3). The amount of plunge can change along the strike of a fold, and a reversal of plunge direction can occur. The axial line is then waved; concave upwards areas are termed depressions and convex upwards areas are known as culminations.

Types of Folding

Anticlines and synclines are symmetrical if both limbs are arranged equally about the axial plane so that the dips on opposing flanks are the same, otherwise they are asymmetrical (Fig. 2.4a and b). In symmetrical folds, the axis is vertical, whereas it is inclined in asymmetrical folds. As folding movements become intensified, overfolds are formed in which both limbs are inclined, together with the axis, in the same direction but at different angles (Fig. 2.4a). In a recumbent fold, the beds have been completely overturned so that one limb is inverted, and the limbs, together with the axial plane, dip at a low angle (Fig. 2.4a).





(b)

Figure 2.2

(a) Block diagram of a non-plunging overturned anticline and syncline, showing various fold elements. (b) A syncline with an anticline to the left, Cape Fold Belt, near George, South Africa.

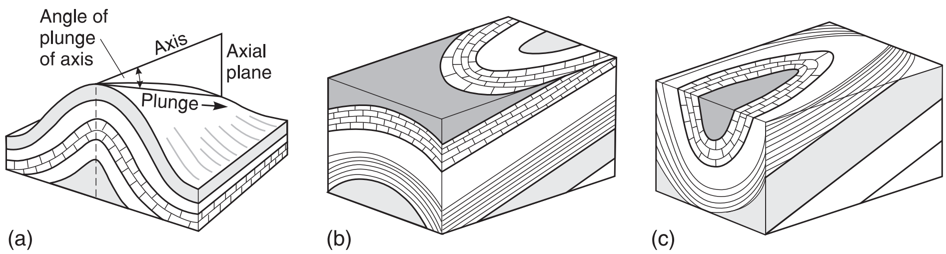


Figure 2.3

(a) Block diagram of an anticlinal fold illustrating plunge. (b) Eroded plunging anticline. (c) Eroded plunging syncline.

If beds that are horizontal, or nearly so, suddenly dip at a high angle, then the feature they form is termed a monocline (Fig. 2.4c). When traced along their strike, monoclines may flatten out eventually or pass into a normal fault; indeed, they often are formed as a result of faulting at depth. Isoclinal folds are those in which both the limbs and the axial plane are parallel (Fig. 2.4d). A fan fold is one in which both limbs are folded (Fig. 2.4e).

Relationships of Strata in Folds

Parallel or concentric folds are those where the strata have been bent into more or less parallel curves in which the thickness of the individual beds remains the same. From Figure 2.5a, it can be observed that, because the thickness of the beds remains the same on folding, the shape of the folds changes with depth and, in fact, they fade out. Parallel folding occurs in competent (relatively strong) beds that may be interbedded with incompetent (relatively weak, plastic) strata.

Similar folds are those that retain their shape with depth. This is accomplished by flowage of material from the limbs into the crest and trough regions (Fig. 2.5b). Similar folds are developed in incompetent strata. However, true similar folds are rare in nature, for most change their shape to some degree along the axial plane. Most folds exhibit both the characteristics of parallel and similar folding.

Most folding is disharmonic in that the shape of the individual folds within the structure is not uniform, with the fold geometry varying from bed to bed. Disharmonic folding occurs in interbedded competent and incompetent strata. Its essential feature is that incompetent horizons display more numerous and smaller folds than the more competent beds enclosing them. It is developed because competent and incompetent beds react differently to stress.

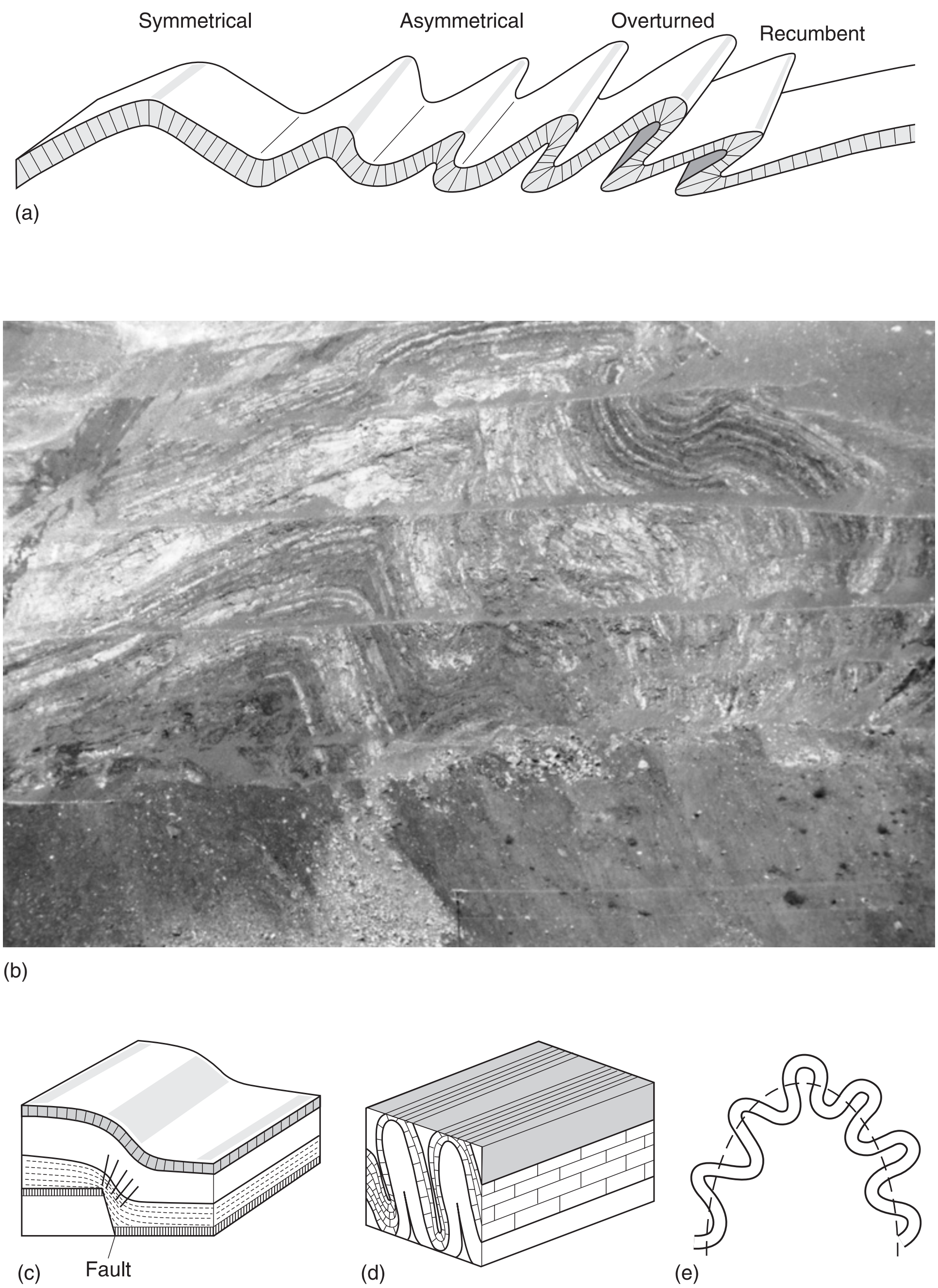


Figure 2.4

(a) Types of folds. (b) An asymmetrical anticline with some overturning near the apex, exposed in an open pit, near Lethbridge, British Columbia. (c) Monoclinal fold. (d) Isoclinal folding. (e) Fan folding.

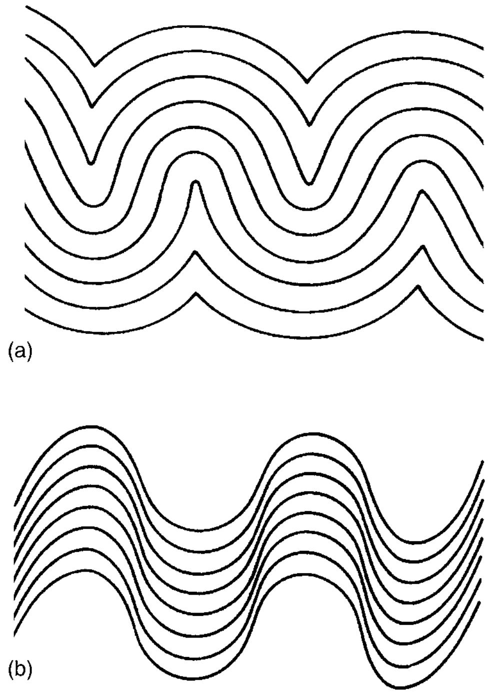


Figure 2.5

(a) Parallel folding. (b) Similar folding.

Zigzag or chevron folds have straight or nearly straight limbs with sharply curved or even pointed hinges (Fig. 2.6). Such folds possess features that are characteristic of both parallel and similar folds in that the strata in their limbs remain parallel, beds may be thinned but they never are thickened, and the pattern of the folding persists with depth. Some bedding slip occurs and gives rise to a small amount of distortion in the hinge regions. The planes about which the beds are bent sharply are called kink planes, and their attitude governs the geometry of the fold. Zigzag folds are characteristically found in thin-bedded rocks, especially where there is a rapid alternation of more rigid beds such as sandstones, with interbedded shales.

Minor Structures Associated with Folding

Cleavage is one of the most notable structures associated with folding and imparts to rocks the ability to split into thin slabs along parallel or slightly sub-parallel planes of secondary origin. The distance between cleavage planes varies according to the lithology of the host rock, that is, the coarser the texture, the further the cleavage planes are apart. Two principal types of cleavage, namely, flow cleavage, and fracture cleavage, have been recognized.

Figure 2.6

Chevron fold in limestone of Miocene age, Kaikuora, South Island, New Zealand.

Flow cleavage occurs as a result of plastic deformation in which internal readjustments involving gliding, granulation and the parallel reorientation of minerals of flaky habit such as micas and chlorite, together with the elongation of quartz and calcite, take place. The cleavage planes are commonly only a fraction of a millimetre apart, and when the cleavage is well developed, the original bedding planes may have partially or totally disappeared. Flow cleavage may develop in deeply buried rocks that are subjected to simple compressive stress, in which case the cleavage planes are orientated normal to the direction in which the stress was acting. As a result, the cleavage planes run parallel to the axial planes of the folds. Many authors equate flow cleavage with true slaty cleavage that is characteristically developed in slates (see Chapter 1).

Fracture cleavage is a parting defined by closely spaced parallel fractures that are usually independent of any planar preferred orientation of mineral boundaries that may be present in a rock mass. It can be regarded as closely spaced jointing, the distance between the planes being measured in millimetres or even in centimetres (Fig. 2.7). Unlike flow cleavage, there is no parallel alignment of minerals, fracture cleavage having been caused by shearing forces. It therefore follows the laws of shearing and develops at an angle of approximately 30° to the axis of maximum principal stress. However, fracture cleavage often runs almost normal to the bedding planes and, in such instances, it has been assumed that it is related to a shear couple. The external stress creates two potential shear fractures but since one of them

Figure 2.7

Fracture cleavage developed in highly folded Horton Flags, Silurian, near Stainforth, North Yorkshire, England. The inclination of the fracture cleavage is indicated by the near-vertical hammer. The other hammer indicates the direction of the bedding.

trends almost parallel to the bedding, it is unnecessary for fractures to develop in that direction. The other direction of potential shearing is the one in which fracture cleavage ultimately develops, and this is facilitated as soon as the conjugate shear angle exceeds 90°. Fracture cleavage is frequently found in folded incompetent strata that lie between competent beds. For example, where sandstone and shale are highly folded, fracture cleavage occurs in the shale in order to fill the spaces left between the folds of the sandstone. However, fracture cleavage need not be confined to the incompetent beds. Where it is developed in competent rocks, it forms a larger angle with the bedding planes than it does in the incompetent strata.

When brittle rocks are distorted, tension gashes may develop as a result of stretching over the crest of a fold or they may develop as a result of local extension caused by drag exerted when beds slip over each other. Those tension gashes that are the result of bending of competent rocks usually appear as radial fractures concentrated at the crests of anticlines that are sharply folded. They represent failure following plastic deformation. Tension gashes formed by differential slip appear on the limbs of folds and are aligned approximately perpendicular to the local direction of extension. Tension gashes are distinguished from fracture cleavage and other types of fractures by the fact that their sides tend to gape. As a result, they often contain lenticular bodies of vein quartz or calcite.

Tectonic shear zones lie parallel to the bedding and appear to be because of displacements caused by concentric folding. Such shear zones generally occur in clay beds with high clay mineral contents. The shear zones range up to approximately 0.5 m in thickness and may extend over hundreds of metres. Each shear zone exhibits a conspicuous principal slip that forms a gently undulating smooth surface. There are two other main displacement shears. The interior of a shear zone is dominated by displacement shears and slip surfaces lying en echelon inclined at 10 to 30° to the *ab* plane (*a* is the direction of movement, *b* lies in the plane of shear and *c* is at right angles to this plane). These give rise to a complex pattern of shear lenses, the surfaces of which are slickensided (i.e. polished and striated). Relative movement between the lenses is complicated, with many local variations. Thrust shears, and possibly fracture cleavage, also have been noted in these shear zones.

Faults

Faults are fractures in crustal strata along which rocks have been displaced (Fig. 2.8). The amount of displacement may vary from only a few tens of millimetres to several hundred kilometres. In many faults, the fracture is a clean break; in others, the displacement is not restricted to a simple fracture, but is developed throughout a fault zone.

The dip and strike of a fault plane can be described in the same way as those of a bedding plane. The angle of hade is the angle enclosed between the fault plane and the vertical. The hanging wall of a fault refers to the upper rock surface along which displacement has occurred, whereas the foot wall is the term given to that below. The vertical shift along a fault plane is called the throw, and the term heave refers to the horizontal displacement. Where the displacement along a fault has been vertical, then the terms downthrow and upthrow refer to the relative movement of strata on opposite sides of the fault plane.

Classification of Faults

A classification of faults can be based on the direction in which movement took place along the fault plane, on the relative movement of the hanging and foot walls, on the attitude of the fault in relation to the strata involved and on the fault pattern. If the direction of slippage along the fault plane is used to distinguish between faults, then three types may be recognized, namely, dip-slip faults, strike-slip faults and oblique-slip faults. In a dip-slip fault, the slippage

Figure 2.8

Fault in strata of the Limestone Group, Lower Carboniferous, near Howick, Northumberland, England.

occurred along the dip of the fault, in a strike-slip fault, it took place along the strike and in an oblique-slip fault, movement occurred diagonally across the fault plane (Fig. 2.9). When the relative movement of the hanging and foot walls is used as a basis of classification, then normal, reverse and wrench faults can be recognized. A normal fault is characterized by the occurrence of the hanging wall on the downthrown side, whereas the foot wall occupies the downthrown side in a reverse fault. Reverse faulting involves a vertical duplication of strata, unlike normal faults where the displacement gives rise to a region of barren ground (Fig. 2.9). In a wrench fault, neither the foot nor the hanging wall have moved up or down in relation to one another (Fig. 2.9). Considering the attitude of the fault to the strata involved, strike faults, dip (or cross) faults and oblique faults can be recognized. A strike fault is one that trends parallel to the beds it displaces, a dip or cross fault is one that follows the inclination of the strata and an oblique fault runs at angle with the strike of the rocks it intersects. A classification based on the pattern produced by a number of faults does not take into account the effects on the rocks involved. Parallel faults, radial faults, peripheral faults, and en echelon faults are among the patterns that have been recognized.

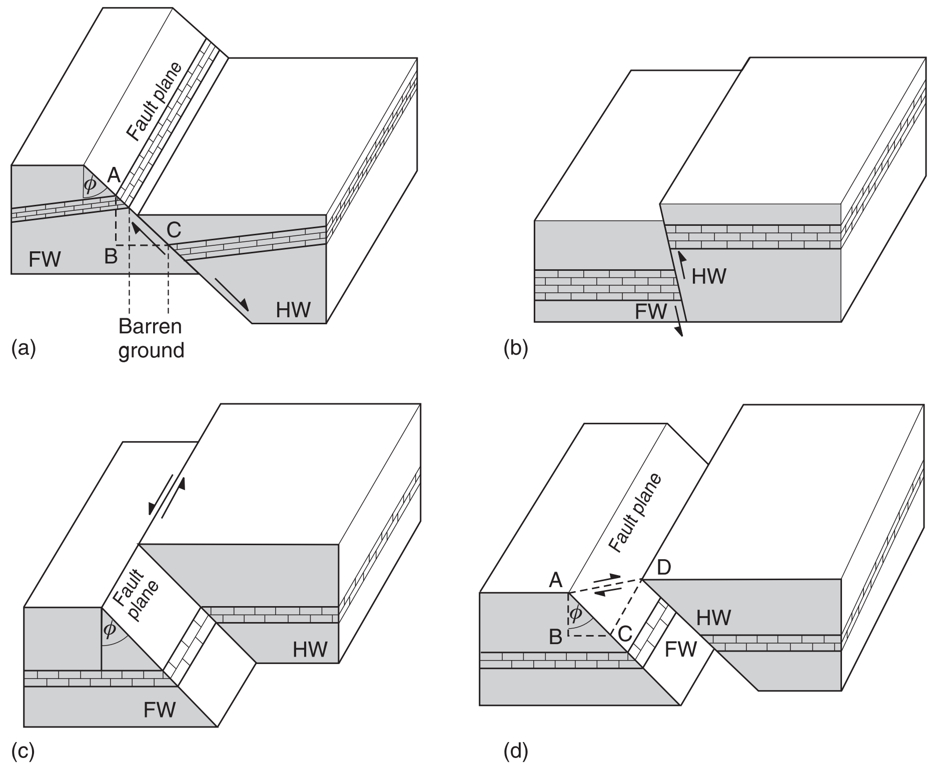


Figure 2.9

Types of faults: (a) normal fault, (b) reverse fault, (c) wrench or strike-slip fault, (d) oblique-slip fault. FW = footwall; HW = hanging wall; AB = throw; BC = heave; ϕ = angle of hade. Arrows show the direction of relative displacement.

In areas that have not undergone intense tectonic deformation, reverse and normal faults generally dip at angles in excess of 45°. Their low-angled equivalents, termed thrusts and lags, respectively, are inclined at less than that figure. Splay faults occur at the extremities of strike-slip faults, and strike-slip faults are commonly accompanied by numerous smaller parallel faults. Sinistral and dextral strike-slip faults can be distinguished in the following manner. When looking across a fault plane, if the displacement on the far side has been to the left, then it is sinistral, whereas if movement has been to the right, the fault is described as dextral.

Normal faults range in linear extension up to, occasionally, a few hundred kilometres in length. Generally, the longer faults do not form single fractures throughout their entirety but consist of a series of fault zones. The net slip on such faults may total over a thousand metres. Normal faults are commonly quite straight in outline but sometimes they may be sinuous or irregular with abrupt changes in strike. When a series of normal faults run parallel to one another with their downthrows all on the same side, the area involved is described as being step faulted (Fig. 2.10). Horsts and rift structures (graben) are also illustrated in Figure 2.10.

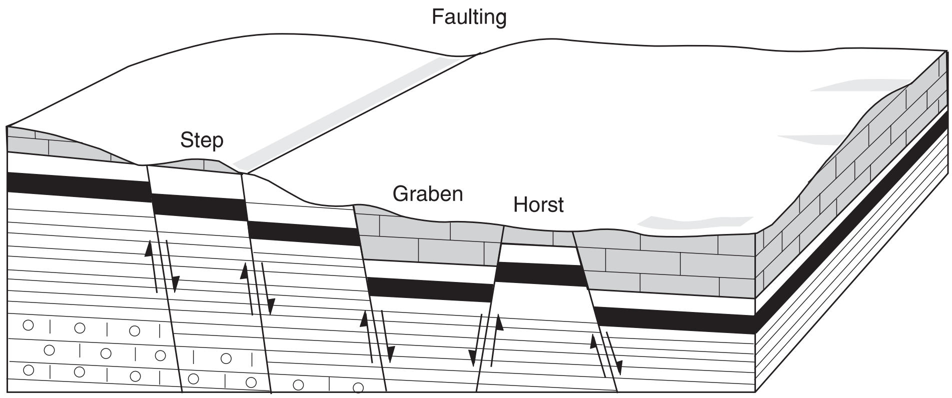


Figure 2.10

Block diagram illustrating step-faulting, and horst and graben structures.

An overthrust is a thrust fault that has a dip of 10° or less, and its net slip measures several kilometres. Overthrusts may be folded or even overturned. As a consequence, when they are subsequently eroded, remnants of the overthrust rocks may be left as outliers surrounded by rocks that lay beneath the thrust. These remnant areas are termed klippe, and the area that separates them from the parent overthrust is referred to as a fenster or a window. The area that occurs in front of the overthrust is called the foreland.

Criteria for the Recognition of Faults

The abrupt ending of one group of strata against another may be caused by the presence of a fault, but abrupt changes also occur at unconformities and intrusive contacts. Nevertheless, it is usually a matter of no great difficulty to distinguish between these three relationships. Repetition of strata may be caused by faulting, that is, the beds are repeated in the same order and dip in the same direction, whereas when they are repeated by folding, they recur in the reverse order and may possess a different inclination (Fig. 2.11a). Omission of strata suggests that faulting has taken place, although such a feature could occur again as a result of unconformity (Fig. 2.11b).

Many features are associated with faulting and, consequently, when found, indicate the presence of a fault. Shear and tension joints frequently are associated with major faults. Shear and tension joints formed along a fault often are referred to as feather joints because of their barb-like appearance. Feather joints may be subdivided into pinnate shear joints and pinnate tension joints. Where pinnate shear planes are closely spaced and involve some displacement, fracture cleavage is developed.

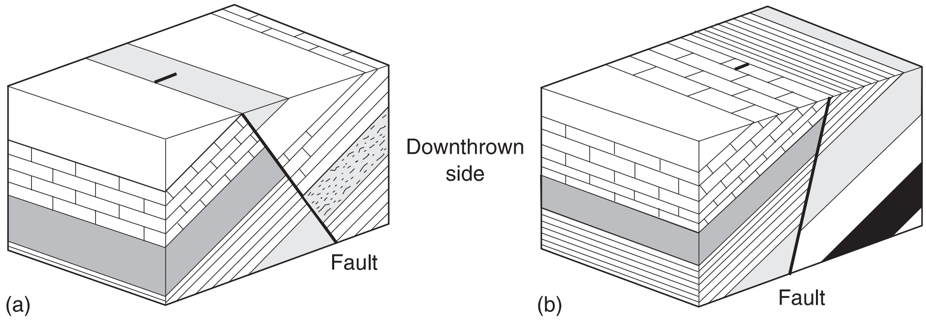


Figure 2.11

(a) Repetition of bed at the surface (fault parallel with the strike and hading against the dip. (b) Omission of bed at the surface (fault parallel with the strike and hading with the dip).

Slickensides are polished striated surfaces that occur on a fault plane and are caused by the frictional effects generated by its movement. Only slight movements are required to form slickensides, and their presence has been noted along shear joints. The striations illustrate the general direction of movement. Very low scarps, sometimes less than a millimetre high, occur perpendicular to the striations and represent small accumulations of material formed as a consequence of the drag effect created by the movement of the opposing block. The shallow face of the scarp points in the direction in which the block moved. Sometimes, two or more sets of slickensides, which usually intersect at an acute angle, may be observed, indicating successive movements in slightly different directions or a sudden deviation in the movement during one displacement.

Intraformational shears, that is, zones of shearing parallel to bedding, are associated with faulting. They often occur in clays, mudstones and shales at the contact with sandstones. Such shear zones tend to die out when traced away from the faults concerned and are probably formed as a result of flexuring of strata adjacent to faults. A shear zone may consist of a single polished or slickensided shear plane, whereas a more complex shear zone may be up to 300 mm in thickness. Intraformational shear zones are not restricted to argillaceous rocks, for instance, they occur in chalk. Their presence means that the strength of the rock along the shear zone has been reduced to its residual value.

As a fault is approached, the strata involved frequently exhibit flexures that suggest that the beds have been dragged into the fault plane by the frictional resistance generated along it. Indeed, along some large dip-slip faults, the beds may be inclined vertically. A related effect is seen in faulted gneisses and schists where a pre-existing lineation is strongly turned into the fault zone and a secondary lineation results.

If the movement along a fault has been severe, the rocks involved may have been crushed, sheared or pulverized. Where shales or clays have been faulted, the fault zone may be occupied by clay gouge. Fault breccias, which consist of a jumbled mass of angular fragments containing a high proportion of voids, occur when more competent rocks are faulted. Crush breccias and crush conglomerates develop when rocks are sheared by a regular pattern of fractures. Movements of greater intensity are responsible for the occurrence of mylonite along a fault zone (see Chapter 1). The ultimate stage in the intensity of movements is reached with the formation of pseudotachylite. This looks like glass.

Although a fault may not be observable, its effects may be reflected in the topography (Fig. 2.12). For example, if blocks are tilted by faulting, a series of scarps are formed. If the rocks on either

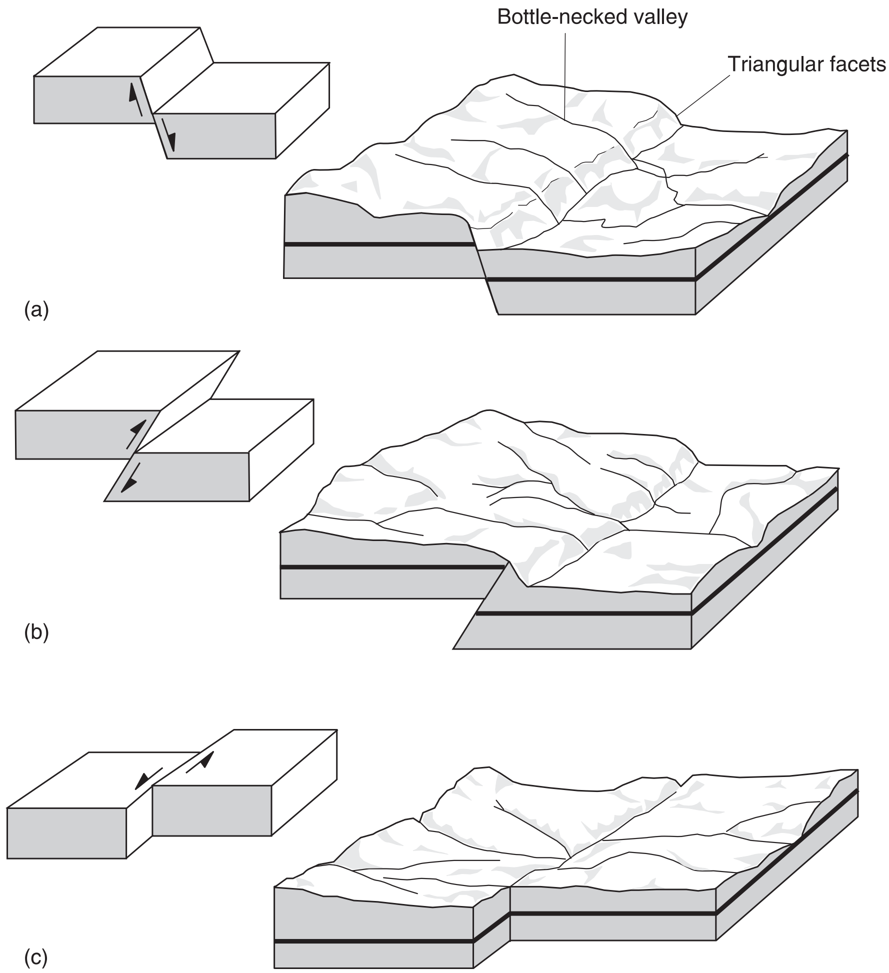


Figure 2.12

(a) Fault scarp formed along normal fault. (b) Reverse fault produces a less distinctive scarp. (c) A strike-slip fault has produced a crush zone that is exploited by a stream. Drainage that once crossed the fault now is offset.

side of a fault are of different strengths, then a scarp may form along the fault as a result of differential erosion. Triangular facets occur along a fault scarp associated with an upland region. They represent the remnants left behind after swift-flowing rivers have cut deep valleys into the fault scarp. Such deeply carved rivers deposit alluvial cones over the fault scarp. Scarplets are indicative of active faults and are found near the foot of mountains, where they run parallel to the base of the range. On the other hand, natural escarpments may be offset by cross faults. Similarly, stream profiles may be interrupted by faults or, in a region of recent uplift, their courses may be relatively straight due to them following faults. Springs often occur along faults. A lake may form if a fault intersects the course of a river and the downthrown block is tilted upstream. Faults may be responsible for the formation of waterfalls in the path of a stream. Sag pools may be formed if the downthrown side settles different amounts along the strike of a fault. However, it must be emphasized that the physiographical features noted above may be developed without the aid of faulting and, consequently, they do not provide a foolproof indication of such stratal displacement.

Faults provide paths of escape and they therefore are frequently associated with mineralization, silicification and igneous phenomena. For example, dykes are often injected along faults.

Discontinuities

A discontinuity represents a plane of weakness within a rock mass across which the rock material is structurally discontinuous. Although discontinuities are not necessarily planes of separation, most in fact are and they possess little or no tensile strength. Discontinuities vary in size from small fissures on the one hand to huge faults on the other. The most common discontinuities are joints and bedding planes (Fig. 2.13). Other important discontinuities are planes of cleavage and schistosity, fissures and faults.

Nomenclature of Joints

Joints are fractures along which little or no displacement has occurred and are present within all types of rocks. At the ground surface, joints may open as a consequence of denudation, especially weathering, or the dissipation of residual stress.

A group of joints that run parallel to each other are termed a joint set, whereas two or more joint sets that intersect at a more or less constant angle are referred to as a joint system. If one set of joints is dominant, then the joints are known as primary joints, and the other set or sets of joints are termed secondary. If joints are planar and parallel or sub-parallel, they are

Figure 2.13

Discontinuities in sandstone of Carboniferous age, near Mansfield, England.

described as systematic; conversely, when their orientation is irregular, they are termed nonsystematic.

Joints can be divided, on the basis of size, into master joints that penetrate several rock horizons and persist for hundreds of metres; major joints that are smaller joints but which are still well-defined structures; and minor joints that do not transcend bedding planes. Lastly, minute fractures occasionally occur in finely bedded sediments, and such micro-joints may be only a few millimetres in size.

Joints may be associated with folds and faults, having developed towards the end of an active tectonic phase or when such a phase has subsided. However, joints do not appear to form parallel to other planes of shear failure such as normal and thrust faults. The orientation of joint sets in relation to folds depends on their size, the type and size of the fold and the thickness and competence of the rocks involved. At times, the orientation of the joint sets can be related directly to the folding and may be defined in terms of the *a, b* and *c* axes of the "tectonic cross" (Fig. 2.14). Those joints that cut the fold at right angles to the axis are called *ac* or cross joints. The *bc* or longitudinal joints are perpendicular to the latter joints, and

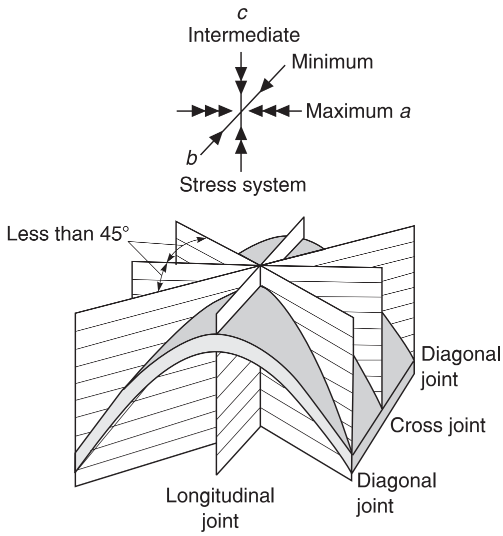


Figure 2.14

Geometric orientation of longitudinal, cross and diagonal joints relative to fold axis and to principal axes of stress.

diagonal or oblique joints run at an angle to both the *ac* and *bc* joints. Diagonal joints are classified as shear joints, whereas *ac* and *bc* joints are regarded as tension joints.

Joints are formed through failure of rock masses in tension, in shear or through some combination of both. Rupture surfaces formed by extension tend to be clean and rough with little detritus. They tend to follow minor lithological variations. Simple surfaces of shearing are generally smooth and contain considerable detritus. They are unaffected by local lithological changes.

Price (1966) contended that the majority of joints are post-compressional structures, formed as a result of the dissipation of residual stress after folding has occurred. Some spatially restricted small joints associated with folds, such as radial tension joints, are probably initiated during folding. Such dissipation of the residual stresses occurs in the immediate neighbourhood of a joint plane so that a very large number of joints need to form in order to dissipate the stresses throughout a large volume.

Joints also are formed in other ways. For example, joints develop within igneous rocks when they cool down, and in wet sediments when they dry out. The most familiar of these are the columnar joints in lava flows, sills and some dykes. The cross joints, longitudinal joints, diagonal joints and flat-lying joints associated with large granitic intrusions have been referred to in Chapter 1. Sheet or mural joints have a similar orientation to flat-lying joints. When they are closely spaced and well developed, they impart a pseudostratification to the host rock. It has been noted that the frequency of sheet jointing is related to the depth of overburden, in other words, the thinner the rock cover, the more pronounced the sheeting. This suggests a connection between the removal of overburden by denudation and the development of sheeting. Indeed, such joints have often developed suddenly during quarrying operations. It may well be that some granitic intrusions contain considerable residual strain energy and that with the gradual removal of load the residual stresses are dissipated by the formation of sheet joints.

Description of Discontinuous Rock Masses

The shear strength of a rock mass and its deformability are influenced very much by the discontinuity pattern, its geometry and how well it is developed. Observation of discontinuity spacing, whether in a field exposure or in a core stick, aids appraisal of rock mass structure. In sedimentary rocks, bedding planes are usually the dominant discontinuities, and the rock mass can be described as shown in Table 2.1. The same boundaries can be used to describe the spacing of joints (Anon, 1977a).

Systematic sets should be distinguished from nonsystematic sets when recording discontinuities in the field. Barton (1978) suggested that the number of sets of discontinuities at any particular location could be described in the following manner:

1. Massive, occasional random joints
2. One discontinuity set
3. One discontinuity set plus random
4. Two discontinuity sets
5. Two discontinuity sets plus random
6. Three discontinuity sets
7. Three discontinuity sets plus random
8. Four or more discontinuity sets
9. Crushed rock, earth-like

Table 2.1. Description of bedding plane and joint spacing (after Anon, 1977a). With kind permission of the Geological Society

Description of bedding plane spacing	Description of joint spacing	Limits of spacing
Very thickly bedded	Extremely wide	Over 2 m
Thickly bedded	Very wide	0.6–2 m
Medium bedded	Wide	0.2–0.6 m
Thinly bedded	Moderately wide	60 mm–0.2 m
Very thinly bedded	Moderately narrow	20–60 mm
Laminated	Narrow	6–20 mm
Thinly laminated	Very narrow	Under 6 mm

As joints represent surfaces of weakness, the larger and more closely spaced they are, the more influential they become in reducing the effective strength of a rock mass. The persistence of a joint plane refers to its continuity. This is one of the most difficult properties to quantify because joints normally continue beyond the rock exposure and, consequently, it is impossible to estimate their continuity. Nevertheless, Barton (1978) suggested that the modal trace lengths measured for each discontinuity set can be described as follows:

Very low persistence	Less than 1 m
Low persistence	1 to 3 m
Medium persistence	3 to10 m
High persistence	10 to 20 m
Very high persistence	Greater than 20 m

Block size provides an indication of how a rock mass is likely to behave, because block size and interblock shear strength determine the mechanical performance of a rock mass under given conditions of stress. The following descriptive terms have been recommended for the description of rock masses in order to convey an impression of the shape and size of blocks of rock material (Barton, 1978):

1. Massive — few joints or very wide spacing
2. Blocky — approximately equidimensional
3. Tabular — one dimension considerably shorter than the other two
4. Columnar — one dimension considerably larger than the other two
5. Irregular — wide variations of block size and shape
6. Crushed — heavily jointed to “sugar cube”

The orientation of the short or long dimensions should be specified in the columnar and tabular blocks, respectively. The actual block size may be described by using the terms given in Table 2.2 (Anon, 1977a).

Discontinuities, especially joints, may be open or closed (Table 2.3). How open they are is of importance in relation to the overall strength and permeability of a rock mass, and this often depends largely on the amount of weathering that the rocks have suffered. Some joints may be partially or completely filled. The type and amount of filling not only influence the effectiveness with which the opposing joint surfaces are bound together, thereby affecting the strength of the rock mass, but also influence permeability. If the infilling is sufficiently thick, the walls of the joint will not be in contact and, hence, the strength of the joint plane will be that of the infill material. Materials such as clay or sand may have been introduced into a joint opening. Mineralization is frequently associated with joints. This may effectively cement a joint; however, in other cases, the mineralizing agent may have altered and weakened the rocks along the joint conduit.

Table 2.2. Block size and equivalent discontinuity spacing (after Anon, 1977). With kind permission of the Geological Society

Term	Block size	Equivalent discontinuity spacings in blocky rock	Volumetric joint count (J_v)* (joints/m^3)
Very large	Over 8 m^3	Extremely wide	Less than 1
Large	0.2–8 m^3	Very wide	1–3
Medium	0.008–0.2 m^3	Wide	3–10
Small	0.0002–0.008 m^3	Moderately wide	10–30
Very small	Less than 0.0002 m^3	Less than moderately wide	Over 30

*After Barton (1978).

The nature of the opposing joint surfaces also influences rock mass behaviour because the smoother they are, the more easily can movement take place along them. However, joint surfaces are usually rough and may be slickensided. Hence, the nature of a joint surface may be considered in relation to its waviness, roughness and the condition of the walls. Waviness and roughness differ in terms of scale and their effect on the shear strength of a joint. Waviness refers to first-order asperities that appear as undulations of the joint surface and are not likely to shear off during movement. Therefore, the effects of waviness do not change with displacements along the joint surface. Waviness modifies the apparent angle of dip but not the frictional properties of the discontinuity. On the other hand, roughness refers to second-order asperities that are sufficiently small to be sheared off during movement. Increased roughness of the discontinuity walls results in an increased effective friction angle along the joint surface. These effects diminish or disappear when infill is present.

A set of terms to describe roughness has been suggested by Barton (1978), based upon two scales of observation, namely, small scale (several centimetres) and intermediate scale

Table 2.3. Description of the aperture of discontinuity surfaces

Anon (1977a)		Barton (1978)		
Description	**Width of aperture**	**Description**		**Width of aperture**
Tight	Zero		Very tight	Less than 0.1 mm
Extremely narrow	Less than 2 mm	Closed	Tight	0.1–0.25 mm
Very narrow	2–6 mm		Partly open	0.25–0.5 mm
Narrow	6–20 mm		Open	0.5–2.5 mm
Moderately narrow	20–60 mm	Gapped	Moderately wide	2.5–10 mm
Moderately wide	60–200 mm		Wide	Over 10 mm
Wide	Over 200 mm		Very wide	10–100 mm
		Open	Extremely wide	100–1000 mm
			Cavernous	Over 1 m

(several metres). The intermediate scale of roughness is divided into stepped, undulating and planar profiles, and the small scale of roughness, superimposed upon the former, includes the rough (or irregular), smooth and slickensided categories. The direction of the slickensides should be noted as shear strength may vary with direction. Barton recognized the classes shown in Figure 2.15.

Joint matching refers to the degree to which the profiles of opposing discontinuity surfaces fit with each other. Various processes such as weathering, shear displacement or loading may affect the extent to which discontinuity surfaces match. Zhao (1997a) introduced a joint matching coefficient, *JMC*, which couples with the joint roughness coefficient, *JRC* (see the following text), which together provide a parameter for correlating joint surface properties. The value of *JMC* ranges from 0 to 1, depending on the matching proportion of the surface area of the discontinuity to the total surface area. A *JMC* value of 1 represents a perfectly matched discontinuity, whereas a value near zero represents minimum surface contact. The values of *JMC* are frequently in the range 0.5–0.8, depending on the degree of alteration along a discontinuity. The aperture of a discontinuity with rough surfaces increases with increasing mismatching and, hence, generally produces a greater amount of closure under compression. Also, a less matched discontinuity tends to be characterized by lower discontinuity stiffness

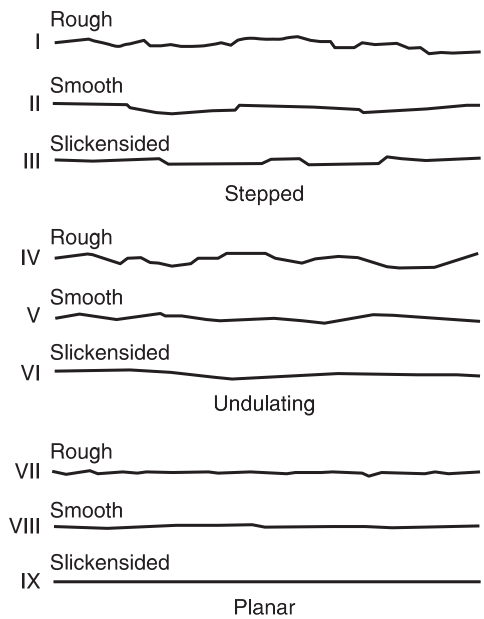


Figure 2.15

Typical roughness profiles and suggested nomenclature. The length of each profile is in the range 1–10 m. The vertical and horizontal scales are equal (after Barton, 1978). With kind permission of Elsevier.

and lower shear strength. Measurement of the *JMC* can be carried out by profiling opposing surfaces of a discontinuity with a profile gauge.

The compressive strength of the rock comprising the walls of a discontinuity is a very important component of shear strength and deformability, especially if the walls are in direct rock-to-rock contact. Weathering (and alteration) frequently is concentrated along the walls of discontinuities, thereby reducing their strength. The weathered material can be assessed in terms of its grade and index tests (see Chapter 3). Samples of wall rock can be tested in the laboratory, not just for strength, but if they are highly weathered, also for swelling and durability.

Seepage of water through rock masses usually takes place via the discontinuities, although in some sedimentary rocks, seepage through the pores also may play an important role. The groundwater level, probable seepage paths and approximate water pressures frequently provide an indication of ground stability or construction problems. Barton (1978) suggested that seepage from open or filled discontinuities could be assessed according to the descriptive scheme shown in Table 2.4.

Table 2.4. Seepage from discontinuities (after Barton, 1978). With kind permission of Elsevier

	Open discontinuities	**Filled discontinuities**
Seepage rating	**Description**	**Description**
(1)	The discontinuity is very tight and dry; water flow along it does not appear possible.	The filling material is heavily consolidated and dry; significant flow appears unlikely due to very low permeability.
(2)	The discontinuity is dry with no evidence of water flow.	The filling materials are damp, but no free water is present.
(3)	The discontinuity is dry but shows evidence of water flow, i.e. rust staining, etc.	The filling materials are wet; occasional drops of water.
(4)	The discontinuity is damp but no free water is present.	The filling materials show signs of outwash, continuous flow of water (estimate l min^{-1}).
(5)	The discontinuity shows seepage, occasional drops of water but no continuous flow.	The filling materials are washed out locally; considerable water flow along outwash channels (estimate l min^{-1} and describe pressure, i.e. low, medium, high).
(6)	The discontinuity shows a continuous flow of water (estimate l min^{-1} and describe pressure, i.e. low, medium, high).	The filling materials are washed out completely; very high water pressures are experienced, especially on first exposure (estimate l min^{-1} and describe pressure).

Strength of Discontinuous Rock Masses and its Assessment

The strength of a discontinuous rock mass is governed by the strength of the intact blocks and the freedom of the blocks to rotate and slide under different stress conditions (Hoek and Brown, 1997). This freedom depends on the shape of the blocks and the condition of the surfaces that separate them. Discontinuities in a rock mass reduce its effective shear strength, at least in a direction parallel to the discontinuities. Hence, the strength of discontinuous rocks is highly anisotropic. Discontinuities offer no resistance to tension, whereas they offer high resistance to compression. Nevertheless, they may deform under compression if there are crushable asperities, compressible filling or apertures along the joint, or if the wall rock is altered. When a jointed rock mass undergoes shearing, this may be accompanied by dilation, especially at low pressures, and small shear displacements probably occur as shear stress builds up.

Barton (1976) proposed the following empirical expression for deriving the shear strength, τ, along joint surfaces:

$$\tau = \sigma_n \tan (JRC \log_{10}(JCS/\sigma_n) + \phi_b) \quad (2.1)$$

where σ_n is the effective normal stress, *JRC* is the joint roughness coefficient, *JCS* is the joint wall compressive strength, and ϕ_b is the basic friction angle. According to Barton, the values of *JRC* range from 0 to 20, from the smoothest to the roughest surface (Fig. 2.16). The *JRC* is constant only for a fixed joint length. Generally, longer profiles (of the same joint) have lower *JRC* values. Indeed, Barton and Bandis (1980) suggested that mobilization of peak strength along a joint surface seems to be a measure of the distance the joint has to be displaced such that asperities are brought into contact. The *JCS* is equal to the unconfined compressive strength of the rock if the joint is unweathered. This may be reduced by up to 75% when the walls of the joints are weathered. Both these factors are related because smooth-walled joints are less affected by the value of *JCS*, as failure of asperities plays a less important role. The smoother the walls of the joints, the more significant is the part played by its mineralogy, ϕ_b. The experience gained from rock mechanics indicates that under low effective normal stress levels, such as those that occur in engineering, the shear strength of joints can vary within relatively wide limits. The maximum effective normal stress acting across joints and considered critical for stability lies, according to Barton, in the range 0.1–2.0 MPa. Zhao (1997b) maintained that the Barton criterion (Eq. 2.1) tends to overpredict the shear strength of discontinuities and that it should be modified to take account of the *JMC* as follows:

$$\tau = \sigma_n \tan [JRC - JMC \log_{10}(JCS/\sigma_n) + \phi_b] \quad (2.2)$$

The shear strength of open discontinuities that are occupied by soft material may be significantly less than if the opposing surfaces of the discontinuities were in close contact. In the case

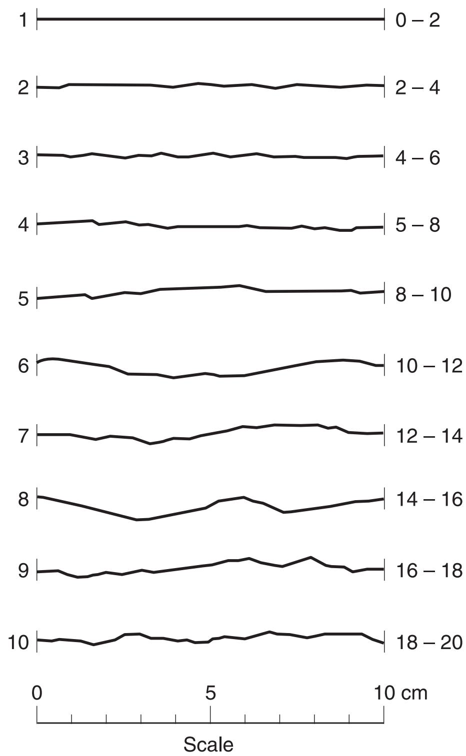


Figure 2.16

Roughness profiles and corresponding range of JRC values associated with each one (after Barton, 1976). With kind permission of Elsevier.

of a rough or undulating discontinuity surface, the thickness of the soft filling has to exceed the amplitude of the undulations if the shear strength is to be reduced to that of the filling.

Discontinuities and Rock Quality Indices

Several attempts have been made to relate the numerical intensity of discontinuities to the quality of unweathered rock masses and to quantify their effect on deformability. For example, the concept of rock quality designation, *RQD*, was introduced by Deere (1964). It is based on the percentage core recovery when drilling rock with NX (57.2 mm) or larger-diameter diamond-core drills. Assuming that a consistent standard of drilling can be maintained, the percentage of solid core obtained depends on the strength and number of discontinuities in the rock mass concerned. The *RQD* is the sum of the core sticks in excess of 100 mm, expressed as a percentage of the total length of core drilled. However, the *RQD* does not take account of the joint opening and condition, a further disadvantage being that with discontinuity spacing greater than 100 mm, the quality is excellent, irrespective of the actual spacing (Table 2.5).

Table 2.5. Classification of rock quality in relation to the incidence of discontinuities

Quality classification	RQD (%)	Fracture frequency per metre	Mass factor (*j*)	Velocity ratio (V_{cf}/V_{cl})
Very poor	0–25	Over 15		0.0–0.2
Poor	25–50	15–8	Less than 0.2	0.2–0.4
Fair	50–75	8–5	0.2–0.5	0.4–0.6
Good	75–90	5–1	0.5–0.8	0.6–0.8
Excellent	90–100	Less than 1	0.8–1.0	0.8–1.0

This particular difficulty can be overcome by using the fracture spacing index. This simply refers to the frequency per metre, with which fractures occur within a rock mass (Table 2.5).

The effect of discontinuities in a rock mass can be estimated by comparing the in situ compressional wave velocity, V_{cf}, with the laboratory sonic velocity, V_{cl}, of an intact core sample obtained from the same rock mass. This gives the velocity ratio V_{cf}/V_{cl}. The difference in these two velocities is caused by the discontinuities that exist in the field. For a high-quality massive rock with only a few tight joints, the velocity ratio approaches unity. As the degree of jointing and fracturing becomes more severe, the velocity ratio is reduced (Table 2.5). The sonic velocity is determined for the core sample in the laboratory under an axial stress equal to the computed overburden stress at the depth from which the rock material was taken, and at a moisture content equivalent to that of the in situ rock. The field seismic velocity is determined preferably by uphole or crosshole seismic measurements in drillholes or test adits, since by using these measurements it is possible to explore individual homogeneous zones more precisely than by surface refraction surveys.

An estimate of the numerical value of the deformation modulus of a jointed rock mass can be obtained from various in situ tests (see Chapter 7). The values derived from such tests are always smaller than those determined in the laboratory from intact core specimens. The more heavily the rock mass is jointed, the larger the discrepancy between the two values. Thus, if the ratio between these two values of deformation modulus is obtained from a number of locations on a site, the engineer can evaluate the rock mass quality. Accordingly, the concept of the rock mass factor, *j*, was introduced by Hobbs (1975), who defined it as the ratio of deformability of a rock mass to that of the intact rock (Table 2.5).

Recording Discontinuity Surveys

Before a discontinuity survey commences, the area in question must be mapped geologically to determine rock types and delineate major structures. It is only after becoming familiar with the geology that the most efficient and accurate way of conducting a discontinuity survey can be devised. A comprehensive review of the procedure to be followed during a discontinuity survey has been provided by Barton (1978) and by Priest (1993).

One of the most widely used methods for collecting discontinuity data is simply by direct measurement on the ground. A direct survey can be carried out subjectively in that only those structures that appear to be important are measured and recorded. In a subjective survey, the effort can be concentrated on the apparently significant joint sets. Nevertheless, there is a risk of overlooking sets that might be important. Conversely, in an objective survey, all discontinuities intersecting a fixed line or area of the rock face are measured and recorded.

Several methods have been used for carrying out direct discontinuity surveys. In the fracture set mapping technique, all discontinuities occurring in zones of 6 m by 2 m, spaced at 30-m intervals along the face, are recorded. Alternatively, using a series of line scans provides a satisfactory method of joint surveying. The technique involves extending a metric tape across an exposure, levelling the tape and then securing it to the face. Two other scan lines are set out as near as possible at right angles to the first, one more or less vertical and the other horizontal. The distance along a tape at which each discontinuity intersects is noted, as is the direction of the pole to each discontinuity (this provides an indication of the dip direction). The dip of the pole from the vertical is recorded as this is equivalent to the dip of the plane from the horizontal. The strike and dip directions of discontinuities in the field can be measured with a compass and the amount of dip with a clinometer. Measurement of the length of a discontinuity provides information on its continuity. It has been suggested that measurements should be taken over distances of about 30 m, and to ensure that the survey is representative, the measurements should be continuous over that distance. A minimum of at least 200 readings per locality is recommended to ensure statistical reliability. A summary of the other details that should be recorded about discontinuities is given in Figure 2.17.

The information gathered by the scanline method can be supplemented with data from orientated cores from drillholes. The value of data on discontinuities gathered from orientated cores from drillholes depends in part on the quality of the rock concerned, in that poor-quality rock is likely to be lost during drilling. However, it is impossible to assess the persistence, degree of separation or the nature of the joint surfaces. What is more, infill material, especially if it is soft, is not recovered by the drilling operations.

Core orientation can be achieved by using a core orientator or by integral sampling (Fig. 2.18a and b, respectively). In a core orientator, the teeth clamp the instrument in position inside the core barrel until released by pressure on the conical probe. The housing contains a soft aluminium ring against which a ball bearing is indented by pressure from the conical probe, thus marking the bottom of the

Discontinuity survey data sheet

General information

Seq. no. ______ Site ______ Date (Day Month Year) ______ Operator ______ Discontinuity data sheet No. ____ of ____

Nature and orientation of discontinuity

Chainage or No.	Type	Dip	Dip direction	Persistence	Aperture	Nature of infilling	Consistency of infilling	Surface roughness	Trend of lineation	Waviness wavelength	Waviness amplitude	Water/flow	Remarks

Type
0 Fault zone
1 Fault
2 Joint
3 Cleavage
4 Schistosity
5 Shear
6 Fissure
7 Tension crack
8 Foliation
9 Bedding

Dip, Dip direction and trend of lineation (expressed in degrees)

Persistence (expressed in metres)

Aperture
1 Wide (>200 mm)
2 Mod. wide (60–200 mm)
3 Mod. narrow (20–60 mm)
4 Narrow (6–20 mm)
5 Very narrow (2–6 mm)
6 Ext. narrow (<2 mm)
7 Tight

Nature of infilling
1 Clean
2 Surface staining
3 Non-cohesive
4 Inactive clay or clay matrix
5 Swelling clay or clay matrix
6 Cemented
7 Chlorite, talc or gypsum
8 Others – specify

Compressive strength of infilling
1 Very soft (<40 kPa)
2 Soft (40–80 kPa)
3 Firm (80–150 kPa)
4 Stiff (150–300 kPa)
5 Very stiff (300–500 kPa)
6 Hard/v. weak (600–1250 kPa)
7 Weak (1.25–5 MPa)
8 Mod. weak (5–12.5 MPa)
9 Mod. strong (12.5–50 MPa)
10 Strong (50–100 MPa)
11 Very strong (100–200 MPa)
12 Ext. strong (>200 MPa)

Roughness
1 Polished
2 Slickensided
3 Smooth
4 Rough
5 Defined ridges
6 Small steps
7 Very rough

Waviness (express wavelength and amplitude in metres)

Water
1 Dry
2 Seepage

Flow
3 <10 ml/s
4 10–100 ml/s
5 0.1–1 l/s
6 1–10 l/s
7 10–100 l/s
8 >100 l/s

Figure 2.17

Discontinuity survey data sheet (after Anon, 1977a). With kind permission of the Geological Society.

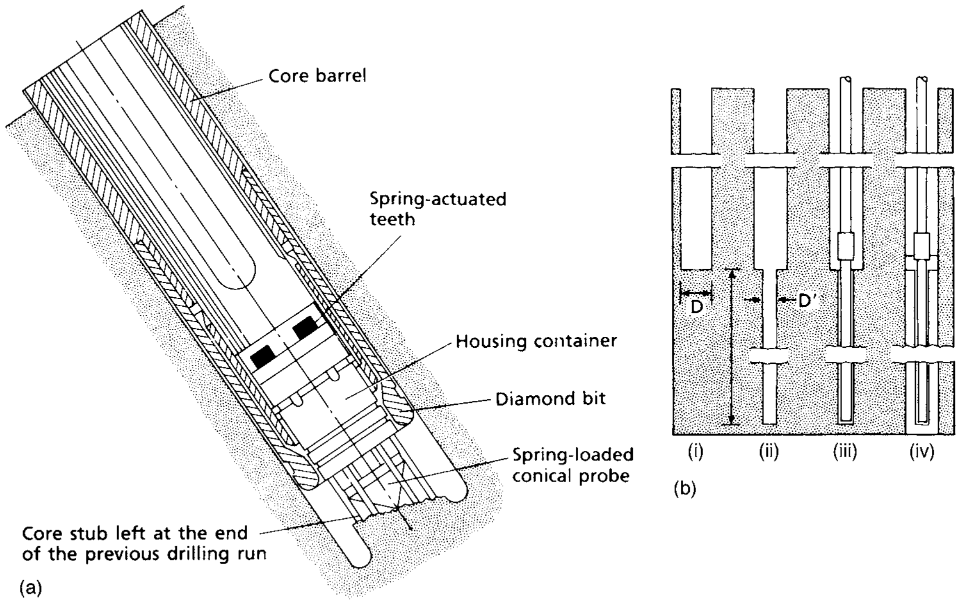


Figure 2.18

(a) Details of a core orientator. The probes take up the profiles of the core stub left by the previous drilling run and are locked in position when the spring loaded cone is released. (b) Stages of the integral sampling method.

hole position. The probe is released by pressure against the core stub and, when released, locks the probe in position and releases the clamping teeth to allow the instrument to ride up inside the barrel ahead of the core entering the barrel. In the first stage of integral sampling, a drillhole (diameter D) is drilled to a depth where the integral sample is to be obtained, then another hole (diameter D′) coaxial with the former and with the same length as the required sample is drilled, into which a reinforcing bar is placed. The bar then is grouted to the rock mass. Drilling is then resumed to obtain the integral sample. The method has been used with success in all types of rock masses, from massive to highly weathered varieties, and provides information on the spacing and orientation, as well as the opening and infilling of discontinuities.

Drillhole inspection techniques include the use of drillhole periscopes, drillhole cameras or closed-circuit television. The drillhole periscope affords direct inspection and can be orientated from outside the hole. However, its effective use is limited to about 30 m. The drillhole camera also can be orientated prior to photographing a section of the wall of a drillhole. The television camera provides a direct view of the drillhole, and a recording can be made on videotape. These three systems are limited in that they require relatively clear conditions and, hence, may be of little use below the water table, particularly if the water in the drillhole is murky. The televiewer produces an acoustic picture of the drillhole wall. One of its advantages is that drillholes need not be flushed prior to its use.

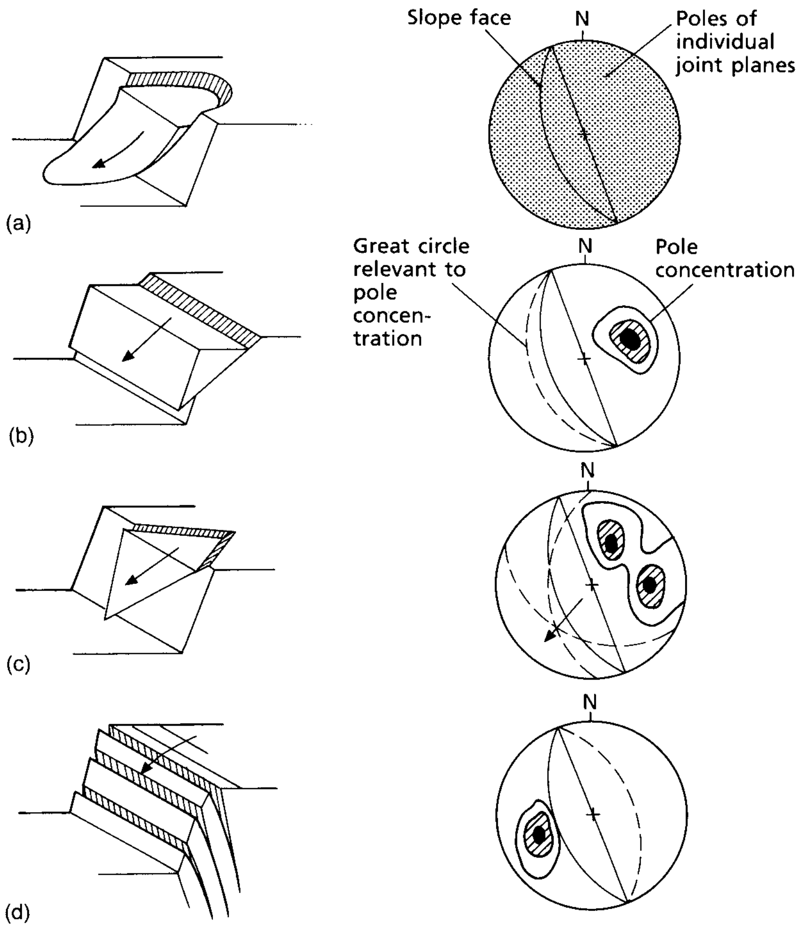


Figure 2.19

Representation of structural data concerning four possible slope failure modes plotted on equal area stereonets as poles, which are contoured to show relative concentration, and great circles. (a) Circular failure in heavily jointed rock with no identifiable structural pattern. (b) Plane failure in highly ordered structure such as slate. (c) Wedge failure on two intersecting sets of joints. (d) Toppling failure caused by steeply dipping joints (after Hoek and Bray, 1981). With kind permission of the Institute of Materials, Minerals and Mining.

Many data relating to discontinuities can be obtained from photographs of exposures. Photographs may be taken looking horizontally at the rock mass from the ground, or they may be taken from the air looking vertically, or occasionally obliquely, down at the outcrop. These photographs may or may not have survey control. Uncontrolled photographs are taken using hand-held cameras. Stereo-pairs are obtained by taking two photographs of the same face from positions about 5% of the distance of the face apart, along a line parallel to the face. Delineation of major discontinuity patterns and preliminary subdivision of the face into structural zones can be made from these photographs. Unfortunately, data cannot be transferred with accuracy from them onto maps and plans. Conversely, discontinuity data can be located accurately on maps and plans by using controlled photographs. Controlled photographs are obtained by aerial photography with complementary ground control or by ground-based phototheodolite surveys. Aerial and ground-based photographs are usually taken with panchromatic film but the use of colour and infrared techniques is becoming more popular. Aerial photographs, with a suitable scale, have proved useful in the investigation of discontinuities. Photographs taken with a phototheodolite also can be used with a stereo-comparator, which produces a stereoscopic model. Measurements of the locations or points in the model can be made with an accuracy of approximately 1 in 5000 of the mean object distance. As a consequence, a point on a face photographed from 50 m can be located to an accuracy of 10 mm. In this way the frequency, orientation and continuity of discontinuities can be assessed. Such techniques prove particularly useful when faces that are inaccessible or unsafe have to be investigated.

Recording Discontinuity Data

Data from a discontinuity survey are usually plotted on a stereographic projection. The use of spherical projections, commonly the Schmidt or Wulf net, means that traces of the planes on the surface of the “reference sphere” can be used to define the dips and dip directions of discontinuity planes. In other words, the inclination and orientation of a particular plane is represented by a great circle or a pole, normal to the plane, which are traced on an overlay placed over the stereonet. The method whereby great circles or poles are plotted on a stereogram has been explained by Hoek and Bray (1981). When recording field observations of the direction and amount of dip of discontinuities, it is convenient to plot the poles rather than the great circles. The poles then can be contoured in order to provide an expression of orientation concentration. This affords a qualitative appraisal of the influence of the discontinuities on the engineering behaviour of the rock mass concerned (Fig. 2.19).

Chapter 3

Surface Processes

All landmasses are continually being worn away or denuded by weathering and erosion, the agents of erosion being the sea, rivers, wind and ice. The detrital products resulting from denudation are transported by water, wind, ice or the action of gravity, and are ultimately deposited. In this manner, the surface features of the Earth are gradually, but constantly, changing. As landscapes are developing continually, it is possible to distinguish the successive stages of their evolution. These stages have been termed youth, maturity and senility. However, the form of landscape that arises during any one of these stages is conditioned partly by the processes of denudation to which the area is subjected, and partly by the structure of the rocks on which the landforms are being developed. Earth movements and type of climate also play a significant role in landscape development.

Weathering

The process of weathering represents an adjustment of the minerals of which a rock is composed to the conditions prevailing on the surface of the Earth. As such, weathering of rocks is brought about by physical disintegration, chemical decomposition and biological activity. It weakens the rock fabric and exaggerates any structural weaknesses, all of which further aid the breakdown processes. A rock may become more friable as a result of the development of fractures both between and within mineral grains. The agents of weathering, unlike those of erosion, do not themselves provide for the transportation of debris from the surface of a rock mass. Therefore, unless the rock waste is otherwise removed, it eventually acts as a protective cover, preventing further weathering. If weathering is to be continuous, fresh rock exposures must be constantly revealed, which means that the weathered debris must be removed by the action of gravity, running water, wind or moving ice.

Weathering also is controlled by the presence of discontinuities in that they provide access into a rock mass for the agents of weathering. Some of the earliest effects of weathering are seen along discontinuity surfaces. Weathering then proceeds inwards so that the rock mass may develop a marked heterogeneity with corestones of relatively unweathered material within a highly weathered matrix (Fig. 3.1). Ultimately, the whole of the rock mass can be reduced to a residual soil. Discontinuities in carbonate rock masses are enlarged by dissolution, leading to the development of sinkholes and cavities within the rock mass.

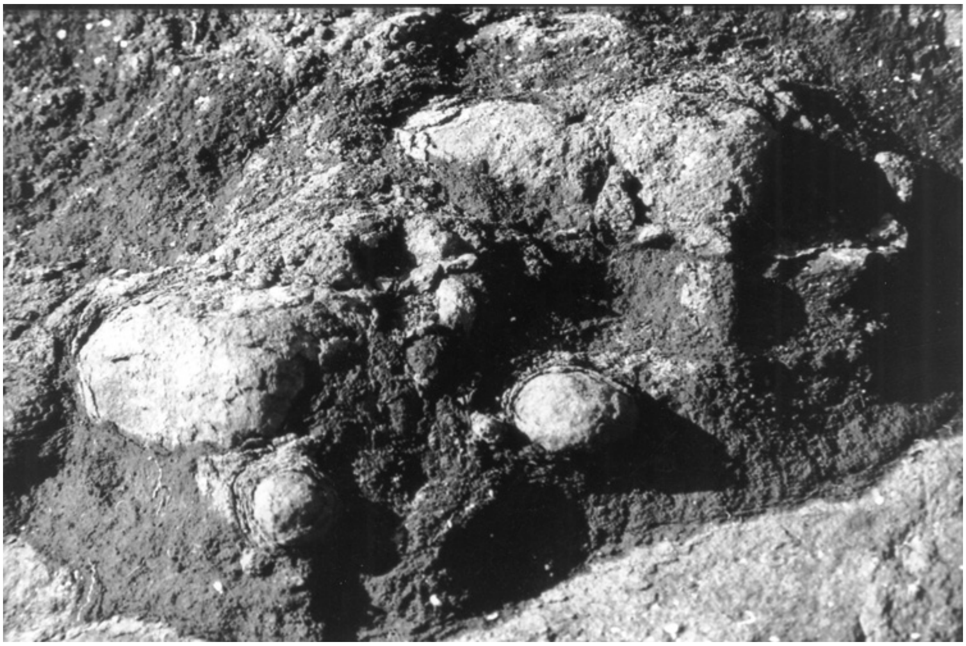


Figure 3.1

Highly weathered basalt, northern Lesotho. Note onion skin weathering.

The rate at which weathering proceeds depends not only on the vigour of the weathering agents but also on the durability of the rock mass concerned. This, in turn, is governed by the mineralogical composition, texture, porosity and strength of the rock on the one hand, and the incidence of discontinuities within the rock mass on the other. Hence, the response of a rock mass to weathering is directly related to its internal surface area and average pore size. Coarse-grained rocks generally weather more rapidly than fine-grained ones. The degree of interlocking between component minerals is also a particularly important textural factor, since the more strongly a rock is bonded together, the greater is its resistance to weathering. The closeness of the interlocking of grains governs the porosity of the rock. This, in turn, determines the amount of water it can hold, and hence, the more porous the rock, the more susceptible it is to chemical attack. Also, the amount of water that a rock contains influences mechanical breakdown, especially in terms of frost action. Nonetheless, deep-weathered profiles usually have been developed over lengthy periods of time. The type and rate of weathering varies from one climatic regime to another. In humid regions, chemical and chemico-biological processes are generally much more significant than those of mechanical disintegration. The degree and rate of weathering in humid regions depends primarily on the temperature and amount of moisture available. An increase in temperature causes an increase in weathering. If the temperature is high, then weathering is extremely active; an increase of 10°C in humid regions more than doubles the rate of chemical reaction. On the other hand, in dry air, chemical decay of rocks takes place very slowly.

Weathering leads to a decrease in density and strength, and to increasing deformability. An increase in the mass permeability frequently occurs during the initial stages of weathering due to the development of fractures, but if clay material is produced as minerals breakdown, then the permeability may be reduced. Widening of discontinuities in carbonate rock masses by dissolution leads to a progressive increase in permeability.

Mechanical Weathering

Mechanical or physical weathering is particularly effective in climatic regions that experience significant diurnal changes of temperature. This does not necessarily imply a large range of temperature, as frost and thaw action can proceed where the range is limited.

Alternate freeze–thaw action causes cracks, fissures, joints and some pore spaces to be widened. As the process advances, angular rock debris is gradually broken from the parent body. Frost susceptibility depends on the expansion in volume that occurs when water moves into the ice phase, the degree of saturation of water in the pore system, the critical pore size, the amount of pore space, and the continuity of the pore system. In particular, the pore structure governs the degree of saturation and the magnitude of stresses that can be generated upon freezing (Bell, 1993). When water turns to ice, it increases in volume by up to 9%, thus giving rise to an increase in pressure within the pores it occupies. This action is further enhanced by the displacement of pore water away from the developing ice front. Once ice has formed, the ice pressures rapidly increase with decreasing temperature, so that at approximately –22 °C, ice can exert a pressure of up to 200 MPa. Usually, coarse-grained rocks withstand freezing better than fine-grained types. The critical pore size for freeze–thaw durability appears to be about 0.005 mm. In other words, rocks with larger mean pore diameters allow outward drainage and escape of fluid from the frontal advance of the ice line and, therefore, are less frost susceptible. Fine-grained rocks that have 5% sorbed water are often very susceptible to frost damage, whereas those containing less than 1% are very durable. Nonetheless, a rock may fail if it is completely saturated with pore water when it is frozen. Indeed, it appears that there is a critical moisture content, which tends to vary between 75 and 96% of the volume of the pores, above which porous rocks fail. The rapidity with which the critical moisture content is reached is governed by the initial degree of saturation.

The mechanical effects of weathering are well displayed in hot deserts, where wide diurnal ranges of temperature cause rocks to expand and contract. Because rocks are poor conductors of heat, these effects are mainly localized in their outer layers where alternate expansion

Figure 3.2

Weathering of granite near Grunau, Namibia.

and contraction creates stresses that eventually rupture the rock. In this way, flakes of rock break away from the parent material, the process being termed exfoliation. The effects of exfoliation are concentrated at the corners and edges of rocks so that their outcrops gradually become rounded (Fig. 3.2). However, in hot semi-arid regions, exfoliation can take place on a large scale with large slabs becoming detached from the parent rock mass. Furthermore, minerals possess different coefficients of expansion, and differential expansion within a polymineralic rock fabric generates stresses at grain contacts and can lead to granular disintegration.

There are three ways whereby salts within a rock can cause its mechanical breakdown: by pressure of crystallization, by hydration pressure, and by differential thermal expansion. Under certain conditions, some salts may crystallize or recrystallize to different hydrates that occupy a larger space (being less dense) and exert additional pressure, that is, hydration pressure. The crystallization pressure depends on the temperature and degree of supersaturation of the solution, whereas the hydration pressure depends on the ambient temperature and relative humidity. Calculated crystallization pressures provide an indication of the potential pressures that may develop during crystallization in narrow closed channels (see Chapter 6). Crystallization of freely soluble salts such as sodium chloride, sodium sulphate or sodium

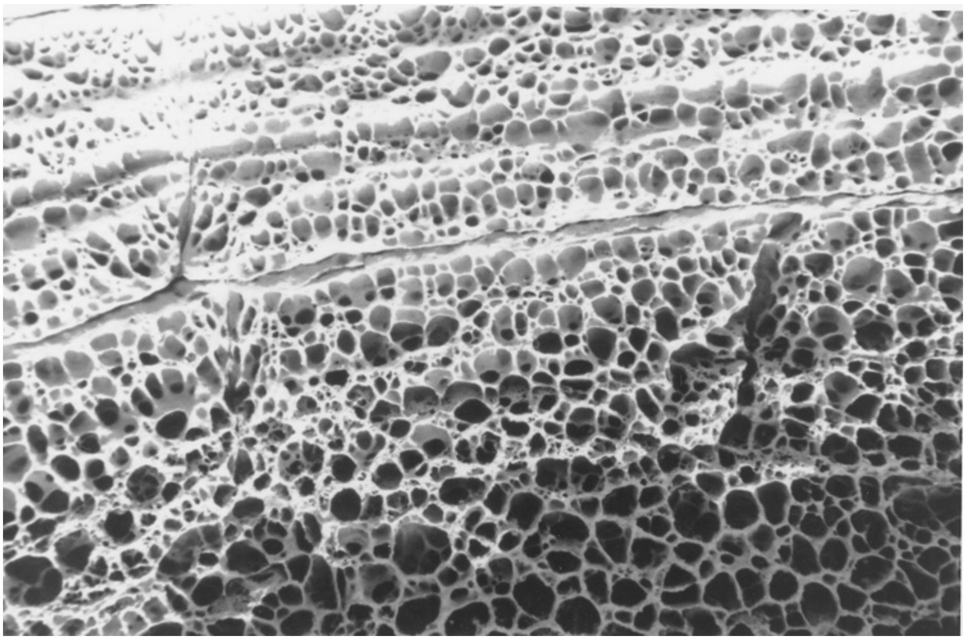


Figure 3.3

Honeycomb weathering in sandstone of Jurassic age, Isle of Skye, Scotland.

hydroxide often leads to the crumbling of the surface of a rock such as limestone or sandstone. Salt action can give rise to honeycomb weathering in porous limestone or sandstone possessing a calcareous cement (Fig. 3.3).

Chemical and Biological Weathering

Chemical weathering leads to mineral alteration and the solution of rocks. Alteration is brought about principally by oxidation, hydration, hydrolysis and carbonation, whereas solution is brought about by acidified or alkalized waters. Chemical weathering also aids rock disintegration by weakening the rock fabric and by emphasizing any structural weaknesses, however slight, that it possesses. When decomposition occurs within a rock, the altered material frequently occupies a greater volume than that from which it was derived and, in the process, internal stresses are generated. If this expansion occurs in the outer layers of a rock, then it eventually causes them to peel off from the parent body.

In dry air, rocks decay very slowly. The presence of moisture hastens the rate of decay, firstly, because water is itself an effective agent of weathering and, secondly, because it holds in solution substances that react with the component minerals of the rock. The most important of these substances are free oxygen, carbon dioxide, organic acids and nitrogen acids.

Free oxygen is an important agent in the decay of all rocks that contain oxidizable substances, iron and sulphur being especially suspect. The rate of oxidation is quickened by the presence of water; indeed, it may enter into the reaction itself, for example, as in the formation of hydrates. However, its role is chiefly that of a catalyst. Carbonic acid is produced when carbon dioxide is dissolved in water, and it may possess a pH value of about 5.7. The principal source of carbon dioxide is not the atmosphere but the air contained in the pore spaces in the soil where its proportion may be a hundred or so times greater than it is in the atmosphere. An abnormal concentration of carbon dioxide is released when organic material decays. Furthermore, humic acids are formed by the decay of humus in soil waters; they ordinarily have pH values between 4.5 and 5.0, but they may occasionally be less than 4.0.

The simplest reactions that take place on chemical weathering are the solution of soluble minerals and the addition of water to substances to form hydrates. Solution commonly involves ionization, for example, this takes place when gypsum and carbonate rocks are weathered. Hydration takes place among some substances, a common example being gypsum and anhydrite:

$$\underset{\text{(anhydrite)}}{CaSO_4} + 2H_2O \rightarrow \underset{\text{(gypsum)}}{CaSO_4.2H_2O}$$

This reaction produces an increase in volume of approximately 6% and, accordingly, causes the enclosing rocks to be wedged further apart. Iron oxides and hydrates are conspicuous products of weathering, usually the oxides are a shade of red and the hydrates yellow to dark brown.

Sulphur compounds are oxidized by weathering. Because of the hydrolysis of the dissolved metal ion, solutions forming from the oxidation of sulphides are acidic. For instance, when pyrite is oxidized initially, ferrous sulphate and sulphuric acid are formed. Further oxidation leads to the formation of ferric sulphate. The formation of anhydrous ferrous sulphate can give rise to a volume increase of about 350%. Very insoluble ferric oxide or hydrated oxide is formed if highly acidic conditions are produced. Sulphuric acid may react with calcite to give gypsum that involves an expansion in volume of around 100%.

Perhaps the most familiar example of a rock prone to chemical attack is limestone. Limestones are chiefly composed of calcium carbonate. Aqueous dissolution of calcium carbonate introduces the carbonate ion into water, that is, CO_3 combines with H to form the stable bicarbonate, H_2CO_3:

$$CaCO_3 + H_2CO_3 \rightarrow Ca(HCO_3)_2$$

In water with a temperature of 25°C, the solubility of calcium carbonate ranges from 0.01 to 0.05 g l^{-1}, depending on the degree of saturation with carbon dioxide. Dolostone is somewhat less soluble than limestone. When limestone is subject to dissolution, any insoluble material present in it remains behind.

Weathering of the silicate minerals is primarily a process of hydrolysis. Much of the silica that is released by weathering forms silicic acid but, when liberated in large quantities, some of it may form colloidal or amorphous silica. Mafic silicates usually decay more rapidly than felsic silicates and, in the process, they release magnesium, iron and lesser amounts of calcium and alkalies. Olivine is particularly unstable, decomposing to form serpentine, which forms talc and carbonates on further weathering. Chlorite is the commonest alteration product of augite (the principal pyroxene) and of hornblende (the principal amphibole).

When subjected to chemical weathering, feldspars decompose to form clay minerals, which are, consequently, the most abundant residual products. The process is brought about by the hydrolysing action of weakly carbonated water that leaches the bases out of the feldspars and produces clays in colloidal form. The alkalies are removed in solution as carbonates from orthoclase (K_2CO_3) and albite (Na_2CO_3), and as bicarbonate from anorthite [$Ca(HCO_3)_2$]. Some silica is hydrolysed to form silicic acid. Although the exact mechanism of the process is not fully understood, the following equation is an approximation towards the truth:

$$\underset{\text{(orthoclase)}}{2KAlSi_3O_6} + 6H_2O + CO_2 \rightarrow \underset{\text{(kaolinite)}}{Al_2Si_2O_5(OH)_4} + 4H_2SiO_4 + K_2CO_3$$

The colloidal clay eventually crystallizes as an aggregate of minute clay minerals. Deposits of kaolin are formed when percolating acidified waters decompose the feldspars contained in granitic rocks.

Clays are hydrated aluminium silicates, and when they are subjected to severe chemical weathering in tropical regimes, notably with wet and dry seasons, they break down to form laterite or bauxite. The process involves the removal of siliceous material, and this is again brought about by the action of carbonated waters. Intensive leaching of soluble mineral matter from surface rocks takes place during the wet season. During the subsequent dry season, groundwater is drawn to the surface by capillary action, and minerals are precipitated there as the water evaporates. The minerals generally consist of hydrated peroxides of iron, and sometimes of aluminium, and very occasionally of manganese. The precipitation of these insoluble hydroxides gives rise to an impermeable lateritic soil. When this point is reached, the formation of laterite ceases as no further leaching can occur. As a consequence, lateritic deposits are usually less than 7 m thick.

Plants and animals play an important role in the breakdown and decay of rocks, indeed their part in soil formation is of major significance. Tree roots penetrate cracks in rocks and gradually wedge the sides apart, whereas the adventitious root system of grasses breaks down small rock fragments to particles of soil size. Burrowing rodents also bring about mechanical disintegration of rocks. The action of bacteria and fungi is largely responsible for the decay of dead organic matter. Other bacteria are responsible, for example, for the reduction of iron or sulphur compounds.

Slaking and Swelling of Mudrocks

Mudrocks are more susceptible to weathering and breakdown than many other rock types. The breakdown of mudrocks starts with exposure, which leads to the opening and development of fissures as residual stress is dissipated, and to an increase in moisture content and softening. The two principal controls on the breakdown of mudrocks are slaking and the expansion of mixed-layer clay minerals. The lithological factors that govern the durability of mudrocks include the degree of induration, the degree of fracturing, the grain size distribution and the mineralogical composition, especially the nature of the clay mineral fraction.

Slaking refers to the breakdown of rocks, especially mudrocks, by alternate wetting and drying. If mudrock is allowed to dry out, air is drawn into the outer pores, and high suction pressures develop. When the mudrock is saturated next, the entrapped air is pressurized as water is drawn into the rock by capillary action. This slaking process causes the internal arrangement of grains to be stressed. Given enough cycles of wetting and drying, breakdown can occur as a result of air breakage, the process ultimately reducing the mudrock involved to tabular-shaped, gravel-size particles. The slake durability test estimates the resistance to wetting and drying of a rock sample, particularly mudstones and rocks that exhibit a certain degree of alteration. In this test, the sample, which consists of ten pieces of rock, each weighing about 40 g, is placed in a test drum, oven dried and then weighed. After this, the drum, with sample, is half immersed in a tank of water and attached to a rotor arm that rotates the drum for a period of 10 min at 20 rev/min (Fig. 3.4). The cylindrical periphery of the drum is formed of a 2-mm sieve mesh so that broken-down material can be lost whilst the test is in progress. After slaking, the drum and the material retained are dried and weighed. The slake durability index is obtained by dividing the weight of the sample retained by its original weight and expressing the answer as a percentage. The following scale is used:

Very low	Under 25%
Low	25 to 50%
Medium	50 to 75%
High	75 to 90%
Very high	90 to 95%
Extremely high	Over 95%

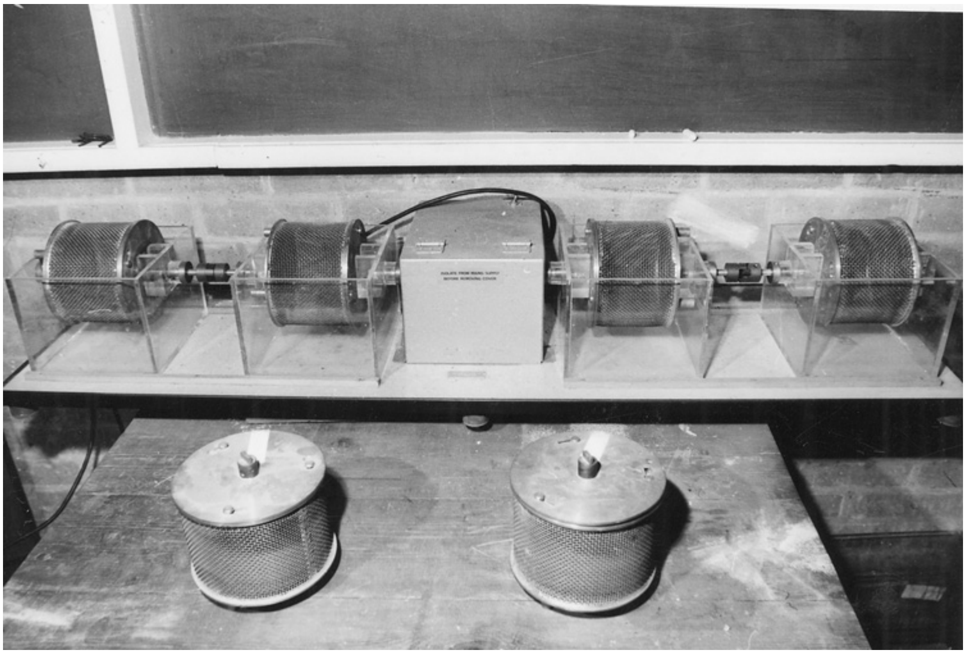


Figure 3.4

The slake-durability apparatus.

Intraparticle swelling (i.e. swelling due to the take up of water – not only between particles of clay minerals but also within them – into the weakly bonded layers between molecular units) of clay minerals on saturation can cause mudrocks to break down where the proportion of such minerals constitutes more than 50% of the rock. The expansive clay minerals such as montmorillonite can expand many times their original volume.

Failure of consolidated and poorly cemented rocks occurs during saturation when the swelling pressure or internal saturation swelling stress, σ_s' developed by capillary suction pressures exceeds their tensile strength. An estimate of σ_s can be obtained from the modulus of deformation, E:

$$E = \sigma_S / \varepsilon_D \tag{3.1}$$

where ε_D is the free-swelling coefficient. The latter is determined by a sensitive dial gauge that records the amount of swelling of an oven-dried core specimen per unit height, along the vertical axis during saturation in water for 12 h, ε_D being obtained as follows:

$$\varepsilon_D = \frac{\text{Change in length after swelling}}{\text{Initial length}} \tag{3.2}$$

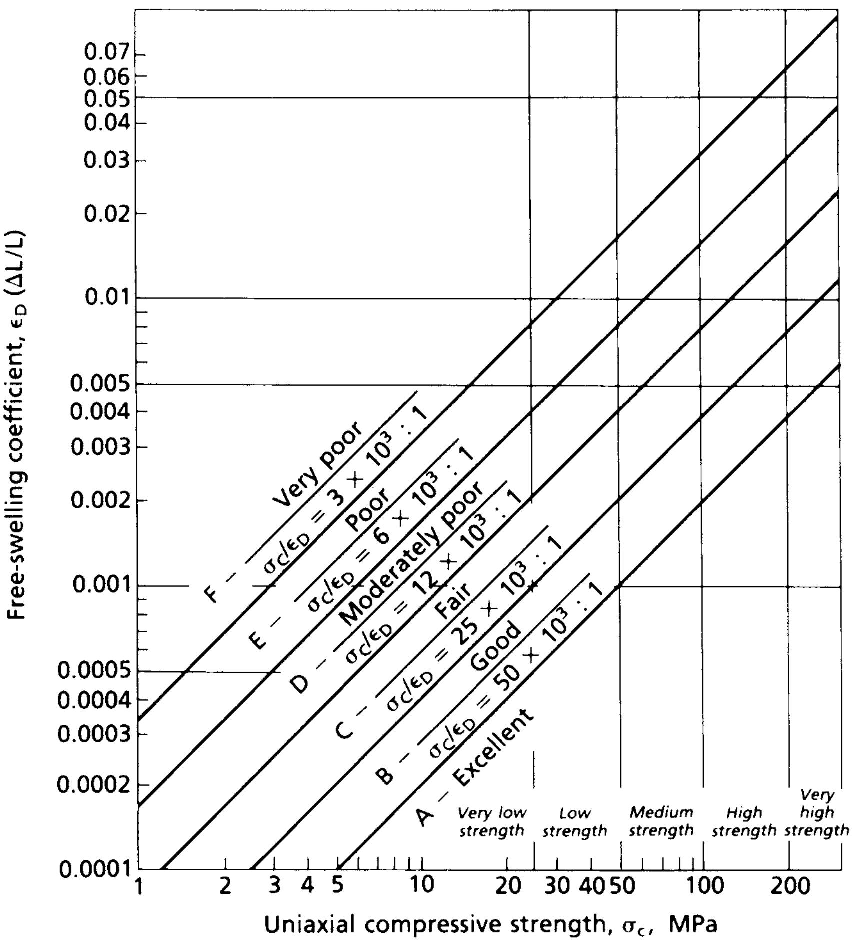


Figure 3.5

Geodurability classification chart for intact rock. Note: (i) ε_D is determined from oven-dried (105°C) to 24-h-saturation condition. (ii) ε_D is plotted as the range and mean of the test results. (iii) strength ratings are according to Bieniawski (1974) (after Olivier, 1979). With kind permission of Elsevier.

Olivier (1979) proposed the geodurability classification, which is based on the free-swelling coefficient and uniaxial compressive strength (Fig. 3.5). This classification was developed to assess the durability of mudrocks.

Engineering Classification of Weathering

The early stages of weathering are usually represented by discoloration of the rock material, which changes from slightly to highly discoloured as the degree of weathering increases.

Because weathering brings about changes in engineering properties, in particular it commonly leads to an increase in porosity with a corresponding reduction in density and strength, these changes being reflected in the amount of discoloration. As weathering proceeds, the rock material becomes increasingly decomposed and/or disintegrated until a soil is ultimately formed. Hence, various stages in the reduction process of a rock to a soil can be recognized.

Numerous attempts have been made to devise engineering classifications of weathered rock and rock masses. Classification schemes have involved quantification of the amount of mineralogical alteration and structural defects in samples with the aid of the petrological microscope. Others have resorted to some combination of simple index tests to provide a quantifiable grade of weathering. Some of the earliest methods of assessing the degree of weathering were based on a description of the character of the rock mass concerned as seen in the field. Such descriptions recognized different grades of weathering and attempted to relate them to engineering performance.

As mineral composition and texture influence the physical properties of a rock, petrographic techniques can be used to evaluate successive stages in mineralogical and textural changes brought about by weathering. Accordingly, Irfan and Dearman (1978) developed a quantitative method of assessing the grade of weathering of granite in terms of its megascopic and microscopic petrography. The megascopic factors included an evaluation of the amount of discoloration, decomposition and disintegration shown by the rock. The microscopic analysis involved assessment of mineral composition and degree of alteration by modal analysis and microfracture analysis. Various chemical changes in rock also have been used to assess the grade of weathering (Kim and Park, 2003). Similarly, physical properties such as bulk density and index tests such as the quick absorption test have been used to distinguish different grades of weathering. A further example of the use of physical tests for the recognition of weathering grades has been provided by Iliev (1967), who developed a coefficient of weathering, K, for granitic rock, based upon the ultrasonic velocities of the rock material according to the expression:

$$K = (V_u - V_w)/V_u \tag{3.3}$$

where V_u and V_w are the ultrasonic velocities of the fresh and weathered rocks, respectively (Table 3.1).

Assessment of the grade of weathering based on a simple description of the geological character of the rock concerned as seen in the field was initially developed by Moye (1955), who proposed a grading system for the degree of weathering found in granite at the Snowy Mountains scheme in Australia. Similar classifications were advanced subsequently that were directed primarily towards the degree of weathering in granitic rocks. Others, working on different rock types, have proposed modified classifications of weathering grade. For example,

Table 3.1. Ultrasonic velocity and grade of weathering

Grade of weathering	Ultrasonic velocity (m s^{-1})	Coefficient of weathering
Fresh	Over 5000	0
Slightly weathered	4000–5000	0–0.2
Moderately weathered	3000–4000	0.2–0.4
Strong weathered	2000–3000	0.4–0.6
Very strongly weathered	Under 2000	0.6–1.0

classifications of weathered chalk and weathered mudstone (marl) have been developed by Ward et al. (1968) and Chandler (1969), respectively. Usually, the grades lie one above the other in a weathered profile developed from a single rock type, the highest grade being at the surface. But this is not necessarily the case in complex geological conditions. Even so, the concept of grade of weathering can still be applied. Such a classification can be used to produce maps showing the distribution of the grade of weathering at particular engineering sites.

Anon (1995) concluded that the most effective schemes for the classification of weathered rock have been those involving the description of the grade of weathering of intact rock or of zones of mass weathering. As far as this report was concerned, it considered that five approaches were required in order to cover different situations and scales. These are summarized in Figure 3.6. The first approach covers the general description of weathering features in rock and forms part of a full description. This description does not involve formal classification but could provide enough information that could be used subsequently for a particular classification purpose. Approach 2 classifies the gradation of weathering of intact rock and is based primarily on strength as determined by simple field tests. Approach 3 is used for rock masses in which the weathering profiles consist of a mixture of relatively strong and weak material. Such a classification can be used to distinguish relatively large zones of different engineering characteristics. The fourth approach was developed for those rock masses in which the scale and heterogeneity of weathering is such that a simple classification scale that incorporates both intact material and rock mass characteristics is appropriate. It was suggested that this approach is likely to be applicable to weaker sedimentary rocks, especially mudrocks. The last approach is for those rock types that weather in a particular way, such as carbonate rocks and some evaporitic deposits.

Movement of Slopes

Soil Creep and Valley Bulging

Creep refers to the slow downslope movement of superficial rock or soil debris, which usually is imperceptible except by observations of long duration. It is a more or less continuous

APPROACH 1 : FACTUAL DESCRIPTION OF WEATHERING (MANDATORY)

Standard descriptions should always include comments on the degree, extent and nature of any weathering effects at material or mass scales. This may allow subsequent classification and provides information fo into zones of like character. Typical indications of weathering include:

- Changes in colour
- Reduction in strength
- Changes in fracture state
- Presence character and extent of weathering products

These features should be described using standard terminolodgy, quantified as appropriate, together with non-standard 'English' descriptors as necessary to describe the results of weathering. At the mass scale, the distribution and proportions of the variously weathered materials (e.g. corestones vs matrix) should be recorded.

Can classification be applied unambiguously? — No — DO NOT CLASSIFY

Yes

Rock is moderately strong or stronger in fresh state

APPROACH 2 : CLASSIFICATION FOR UNIFORM MATERIALS

Grade	Classifier	Typical Characteristics
I	Fresh	• Unchanged from original state
II	Slightly Weathered	• Slight discolouration, slight weakening
III	Moderately Weathered	• Considerably weakened, penetrative discolouration • Large pieces cannot be broken by hand
IV	Highly Weathered	• Large pieces can be broken by hand • Does not readily disaggregate (slake) when dry sample immersed in water
V	Completely Weathered	• Considerably weakened • Slakes • Original texture apparent
VI	Residual Soil	• Soil derived by in situ weathering but retaining none of original texture or fabric

Is a zonal classification appropriate and is there enough information available? — No — Use Rock Mass classification if appropriate

Yes

APPROACH 3 : CLASSIFICATION FOR HETEROGENOUS MASSES

ZONE	PROPORTIONS OF MATERIAL GRADES	TYPICAL CHARACTERISTICS
1	100% G I – III (not necessarily all fresh rock)	Behaves as rock, apply rock mechanics principles to mass assessment and design
2	>90% G I – III <10% G IV – VI	Weak materials along discontinuities. Shear strength, stiffness and permeability affected
3	50 to 90% G I – III 10 to 50% G IV – VI	Rock framework still locked and controls strength and stiffness, matrix controls permeability
4	30 to 50% G I – III 50 to 70% G IV – VI	Rock framework contributes to strength, matrix or weathering products control stiffness and permeability
5	<30% G I – III >70% G IV – VI	Weak grade will control behaviour. Corestones may be significant for investigation and construction
6	100% G IV – VI (not necessarily all residual soil)	May behave as soil although relict fabric may be still be significant

Rock is moderately weak or weaker in fresh state

APPROACH 4 : CLASSIFICATION INCORPORATING MATERIAL AND MASS FEATURES

Grade	Classifier	Typical Characteristics
A	Unweathered	Original strength colour, fracture spacing
B	Partially Weathered	Slightly reduced strength, sligthly closer fracture spacing, weathering penetrating in from fractures, brown oxidation
C	Distinctly Weathered	Further weakened, much closer fracture spacing, grey reduction
D	Destructed	Greatly weakened, mottled, ordered lithorelicts in matrix becoming weakened and disordered, bedding distributed
E	Residual or Reworked	Matrix with occasional altered random or "apparent" lithorelicts bedding destroyed. Classed as reworked when foreign inclusions are present as a result of transportation

APPROACH 5 : SPECIAL CASES

For rocks whose weathering state does not follow the other patterns indicated here, such as karst in carbonates and the particular effect of arid climates

Figure 3.6

Approaches to weathering description and classification (after Anon, 1995). With kind permission of the Geological Society.

process that is distinctly a surface phenomenon and occurs on slopes with gradients somewhat in excess of the angle of repose of the material involved. Similarly to landslip, the principal cause of creep is gravity, although it may be influenced by seasonal changes in temperature, and by swelling and shrinkage in surface rocks. Evidence of soil creep may be found on many soil-covered slopes. For example, evidence of soil creep occurs as small terracettes, downslope tilting of poles, the curving downslope of trees and soil accumulation on the uphill sides of walls.

Solifluction is a form of creep that occurs in cold climates or high altitudes where masses of saturated rock waste move downslope. Generally, the bulk of the moving mass consists of fine debris but blocks of appreciable size also may be moved. Saturation is brought about by rain or melting snow. Moreover, in periglacial regions, water commonly cannot drain into the ground since it is frozen permanently. Solifluction differs from mudflow in that it moves much more slowly, the movement is continuous and it occurs over the whole slope.

Valley bulges consist of folds formed by mass movement of argillaceous material in valley bottoms, the argillaceous material in the sides of the valley being overlain by thick competent rocks (Fig. 3.7). The amplitude of the fold can reach 30 m in those instances where a single anticline occurs along the line of the valley. Alternatively, the valley floor may be bordered by a pair of reverse faults or a belt of small-scale folding. These features have been explained as stress relief phenomena, that is, as stream erosion proceeded in the valley, the excess loading on the sides caused the argillaceous material to be squeezed towards the area of minimum loading. This caused the rocks in the valley to bulge upwards. However, other factors also may be involved in the development of valley bulging, such as high piezometric pressures, swelling clays or shales and rebound adjustments of the stress field due to valley loading and excavation by ice.

The valleyward movement of argillaceous material results in cambering of the overlying competent strata, blocks of which may become detached and move down the valley side. Fracturing of cambered strata produces deep debris-filled cracks or “gulls” that run parallel to the trend of the valley. Some gulls may be several metres wide.

Landslides

Landsliding is one of the most effective and widespread mechanisms by which landscape is developed. It is of great interest to the engineer since an understanding of the causes of landslides should help provide answers relating to the control of slopes, either natural or man-made. An engineer faced with a landslide is interested primarily in curing the harmful effects of the slide. In many instances, the principal cause cannot be removed so that it may be more economical to alleviate the effects continually. Indeed, in most landslides, a number of causes contribute towards movement and any attempt to decide which one finally produced

Figure 3.7

Valley bulging in interbedded shales and thin sandstones of Namurian age revealed during the excavation for the dam for Howden Reservoir in 1933, South Yorkshire, England.

the failure is not only difficult but pointless. Often, the final factor is nothing more than a trigger mechanism that set in motion a mass that was already on the verge of failure.

Landslides represent the rapid downward and outward movement of slope-forming materials, the movement taking place by falling, sliding or flowing, or by some combination of these factors (Griffiths, 2005). This movement generally involves the development of a slip surface between the separating and remaining masses. However, rockfalls, topples and debris flows involve little or no true sliding on a slip surface. The majority of stresses found in most slopes are the gravitational stress from the weight of the material plus the residual stress.

Landslides occur because the forces creating movement, the disturbing forces, M_D, exceed those resisting them, the resisting forces, M_R, that is, the shear strength of the material concerned. In general terms, the stability of a slope may be defined by a factor of safety, F, where:

$$F = M_R/M_D \tag{3.4}$$

If the factor of safety exceeds one, then the slope is stable, whereas if it is less than one, the slope is unstable.

The common force tending to generate movements on slopes is gravity. Over and above this, a number of causes of landslides can be recognized. These were grouped into two categories by Terzaghi (1950a), namely, external causes and internal causes. The former include those mechanisms outside the mass involved, which are responsible for overcoming its internal shear strength, thereby causing it to fail. Internal mechanisms are those within the mass that bring about a reduction of its shear strength to a point below the external forces imposed on the mass by its environment, thereby inducing failure.

An increase in the weight of slope material means that shearing stresses are increased, leading to a decrease in the stability of a slope, which may ultimately give rise to a slide. This can be brought about by natural or artificial (man-made) activity. For instance, removal of support from the toe of a slope, either by erosion or excavation, is a frequent cause of slides, as is overloading the top of a slope. Such slides are external slides in that an external factor causes failure. Other external mechanisms include earthquakes or other shocks and vibrations. Keefer (1984) suggested that an earthquake with a Richter magnitude of 4 probably would not generate landslides, whereas a magnitude of 9.2 would cause land-slidesto take place over an area as large as 500,000 km^2. He further suggested that rockfalls, rock slides, soil falls and soil slides are triggered by the weaker seismic tremors, whereas deep-seated slides and earthflows are generally the result of stronger earthquakes. Materials that are particularly susceptible to earthquake motions include loess, volcanic ash on steep slopes, saturated sands of low density, quickclays and loose boulders on slopes. The most severe losses of life have generally been caused by earthquake-induced land-slides, for example, the one that occurred in 1920 in Kansu Province, China, killed around 200,000 people.

In many parts of the world, marine erosion on many coastlines was halted by the glacio-eustatic lowering of sea level during Pleistocene times and recommenced on subsequent recovery. For example, landslides around the English coast were generally reactivated by rising sea levels some 4,000 to 8,000 years BP. Hutchinson (1992) stated that once the sea level became reasonably constant, erosion continued at a steady pace, giving rise to coastal landslides. A cyclic situation then develops, in which landslide material is removed by the sea and so the cliffs are steepened, leading to further landsliding. Hence, extended periods of slow movement are succeeded by sudden first-time failures. Previously, Hutchinson (1973) had noted that the cliffs developed in London Clay at Warren Point, Isle of Sheppey, have a landslip cycle of approximately 40 years. By contrast, the coastal landslide cycle for the harder Cretaceous rocks forming the Undercliff, Isle of Wight, is about 6,000 years (Hutchinson et al., 1991).

Internal slides are usually caused by an increase of pore water pressures within the slope material, which causes a reduction in the effective shear strength. Indeed, it is generally agreed that in most landslides, groundwater constitutes the most important single contributory cause. Hence, landslides can be triggered by rainfall if some threshold intensity is exceeded so that pore water pressures are increased by a required amount (Olivier et al., 1994). Rises in the levels of water tables because of short-duration intense rainfall or prolonged rainfall of lower intensity are a major cause of landslides (Bell, 1994a). An increase in moisture content also means an increase in the weight of the slope material or its bulk density, which can induce slope failure. Significant volume changes may occur in some materials, notably clays, on wetting and drying out. Not only does this weaken the clay by developing desiccation cracks within it, but the enclosing strata also may be adversely affected. Seepage forces within granular soil can produce a reduction in strength by reducing the number of contacts between grains.

Weathering can effect a reduction in the strength of slope material, leading to sliding. The necessary breakdown of equilibrium to initiate sliding may take decades. In relatively impermeable cohesive soils, the swelling process is probably the most important factor leading to a loss of strength and, therefore, to delayed failure (Meisina, 2004).

A slope in dry coarse soils should be stable, provided its inclination is less than the angle of repose. Slope failure tends to be caused by the influence of water. For instance, seepage of groundwater through a deposit of sand in which slopes exist can cause them to fail. Failure on a slope composed of granular soil involves the translational movement of a shallow surface layer. The slip is often appreciably longer than it is in depth. This is because the strength of granular soils increases rapidly with depth. If, as is generally the case, there is a reduction in the density of the granular soil along the slip surface, the peak strength is reduced ultimately to the residual strength. The soil will continue shearing without further change in volume once it has reached its residual strength. Although shallow slips are common, deep-seated shear slides can occur in granular soils.

In fine soils, slope and height are interdependent, and can be determined when the shear characteristics of the material are known. Because of their moisture-retaining capacity and low permeability, pore water pressures are developed in cohesive soils. These pore water pressures reduce the strength of the soil. Thus, in order to derive the strength of an element of the failure surface within a slope in cohesive soil, the pore water pressure at that point needs to be determined to obtain the total and effective stresses. This effective stress is then used as the normal stress in a shear box or triaxial test to assess the shear strength of the clay concerned. Skempton (1964) showed that on a stable slope in clay, the resistance offered along a slip surface, that is, its shear strength, s, is given by

$$s = c^1 + (\sigma - u) \tan \phi^1 \tag{3.5}$$

where c^1 = cohesion intercept, ϕ^1 = angle of shearing resistance (these are average values around the slip surface and are expressed in terms of effective stress), σ = total overburden pressure and u = pore water pressure. In a stable slope, only part of the total available shear resistance along a potential slip surface is mobilized to balance the total shear force, τ, hence:

$$\Sigma\tau = \Sigma c^1/F + \Sigma\,(\sigma - u)\tan\phi^1/F \tag{3.6}$$

where F is the factor of safety. If the total shear force equals the total shear strength, then a slip is likely to occur (i.e. F = 1.0).

Clay soils, especially in short-term conditions, may exhibit relatively uniform strength with increasing depth. As a result, slope failures, particularly short-term failures, may be comparatively deep-seated, with roughly circular slip surfaces. This type of failure is typical of relatively small slopes. Landslides on larger slopes are often noncircular failure surfaces following bedding planes or other weak horizons.

The factors that determine the degree of stability of steep slopes in hard unweathered crystalline rocks (defined as rocks with unconfined strengths of 35 MPa and over) have been examined by Terzaghi (1962). Terzaghi contended that landsliding in such rocks is largely dependent on the incidence, orientation and nature of the discontinuities present. The value of the angle of shearing resistance required for a stability analysis, ϕ, depends on the type and degree of interlock between the blocks on either side of the surface of sliding. Terzaghi concluded that the critical slope angles for slopes underlain by strong massive rocks with a random joint pattern is about 70°, provided the walls of the joints are not acted on by seepage pressures.

In a bedded and jointed rock mass, if the bedding planes are inclined, the critical slope angle depends on their orientation in relation to the slope and the orientation of the joints (Hoek and Bray, 1981). The relation between the angle of shearing resistance, ϕ, along a discontinuity, at which sliding will occur under gravity, and the inclination of the discontinuity, α, is important. If $\alpha < \phi$, the slope is stable at any angle, whereas if $\phi < \alpha$, then gravity will induce movement along the discontinuity surface, and the slope cannot exceed the critical angle, which has a maximum value equal to the inclination of the discontinuities. It must be borne in mind, however, that rock masses are generally interrupted by more than one set of discontinuities.

Classification of Landslides

Varnes (1978) classified landslides according to the type of materials involved on the one hand and the type of movement undergone on the other (Fig. 3.8). The materials concerned were grouped as rocks and soils. The types of movement were grouped into falls, slides and

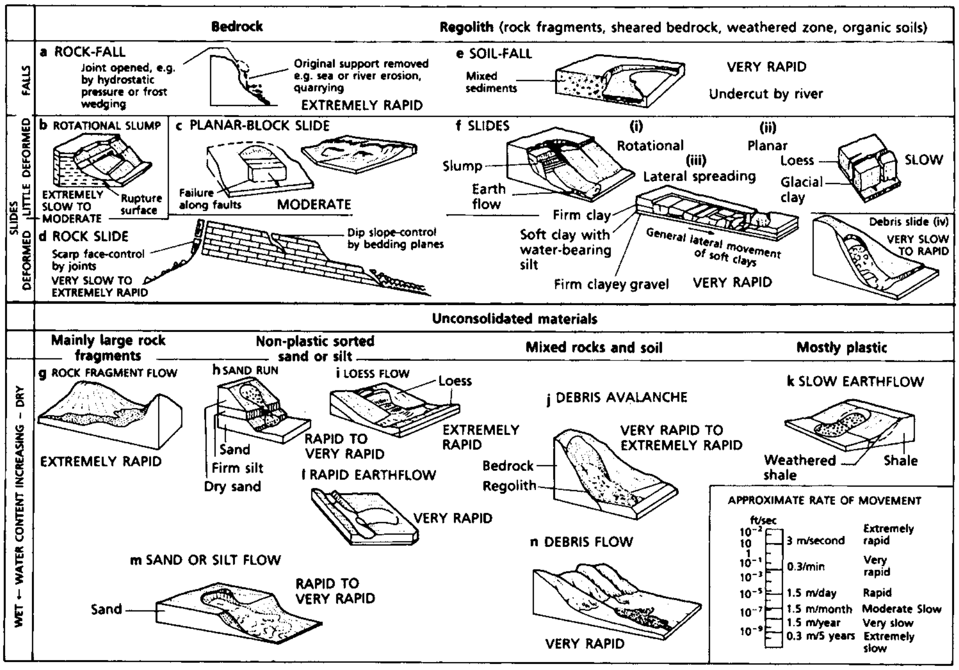


Figure 3.8

A classification of landslides (after Varnes, 1978). With kind permission of the National Academy of Science.

flows; one can, of course, merge into another. Complex slope movements are those in which there is a combination of two or more principal types of movement. Multiple movements are those in which repeated failures of the same type occur in succession.

Falls are very common (Fig. 3.9). The moving mass in a fall travels mostly through the air by free fall, saltation or rolling, with little or no interaction between the moving fragments. Movements are very rapid and may not be preceded by minor movements. In rockfalls, the fragments are of various sizes and are generally broken in the fall. They accumulate at the bottom of a slope as scree. If rockfall is active or very recent, then the slope from which it was derived is scarped. Freeze–thaw action is one of the major causes of rockfall.

Toppling failure of individual blocks is governed by joint spacing and orientation, and is a special type of rockfall that can involve considerable volumes of rock. The condition for toppling is defined by the position of the weight vector in relation to the base of the block involved. If the weight vector, which passes through the centre of gravity of the block, falls outside the base of the block, toppling will occur. Put another way, the condition for stability is that the resultant force must be within the central two thirds of the base of the block. Hydrostatic forces acting

Figure 3.9

Rockfall on the slopes of Table Mountain, Cape Town, South Africa.

at the rear of near-vertical joints greatly affect the direction of the resultant force. The danger of a slope toppling increases with increasing discontinuity angle, and steep slopes in vertically jointed rocks frequently exhibit signs of toppling failure.

In true slides, the movement results from shear failure along one or several surfaces, such surfaces offering the least resistance to movement. The mass involved may or may not experience considerable deformation. One of the most common types of slide occurs in clay soils where the slip surface is approximately spoon-shaped. Such slides are referred to as rotational slides. They are commonly deep-seated (0.15 depth/length < 0.33). Although the slip surface is concave upwards, it seldom approximates to a circular arc of uniform curvature. For instance, if the shear strength of the soil is less in the horizontal than vertical direction, the arc may flatten out; if the soil conditions are reversed, then the converse may apply. What is more, the shape of the slip surface is influenced by the discontinuity pattern of the materials involved (Bell and Maud, 1996).

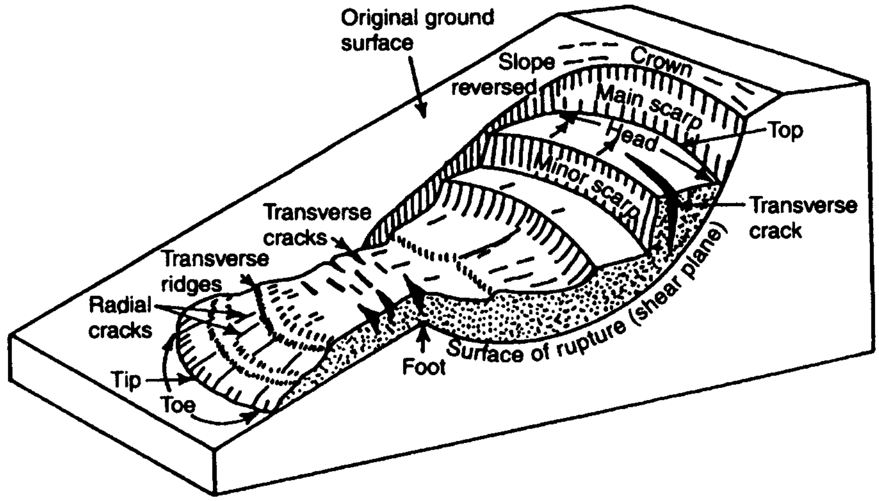


Figure 3.10

Block diagram illustrating the principal features of a rotational slide.

Rotational slides usually develop from tension scars in the upper part of a slope, the movement being more or less rotational about an axis located above the slope (Fig. 3.10). The tension cracks at the head of a rotational slide are generally concentric and parallel to the main scar. When the scar at the head of a rotational slide is almost vertical and unsupported, then further failure will usually occur, it is just a matter of time. As a consequence, successive rotational slides occur until the slope is stabilized. These are retrogressive slides, and they develop in a headward direction. All multiple retrogressive slides have a common basal shear surface in which the individual planes of failure are combined.

Translational slides occur in inclined stratified deposits, the movement occurring along a planar surface, frequently a bedding plane (Fig. 3.11). The mass involved in the movement becomes dislodged because the force of gravity overcomes the frictional resistance along the potential slip surface, the mass having been detached from the parent rock by a prominent discontinuity such as a major joint. Slab slides, in which the slip surface is roughly parallel to the ground surface, are a common type of translational slide. Such a slide may progress almost indefinitely if the slip surface is inclined sufficiently, and the resistance along it is less than the driving force, whereas rotational sliding usually brings equilibrium to an unstable mass. Slab slides can occur on gentler surfaces than rotational slides and may be more extensive.

Rock slides and debris slides are usually the result of a gradual weakening of the bonds within a rock mass and are generally translational in character (Fig. 3.12). Most rock slides are controlled by the discontinuity patterns within the parent rock. Water is seldom an important

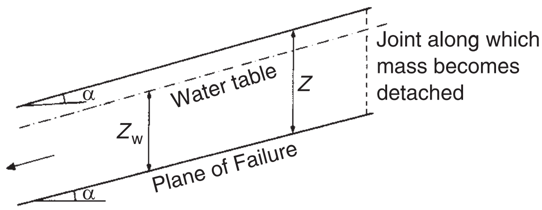


Figure 3.11

A translational slide. Z is the depth of the plane of failure below the surface; Z_w is the depth of the plane of failure below the water table; α is the angle of inclination of the plane of failure and the surface. In a translational slide, it is assumed that the potential plane of failure lies near to and parallel to the surface. The water table is also inclined parallel to the surface. If the water table creates a hydrostatic component of pressure on the slip surface with flow out of the slope, then the factor of safety, F, is: $F = c^1 + (\gamma Z\cos^2\alpha - \gamma_w Z_w)\, Z\tan c\varphi^1/\gamma Z \sin\alpha \cos\alpha$, where γ is the unit weight, γ_w is the unit weight of water and c^1 and φ^1 are the effective cohesion and angle of shearing resistance, respectively.

Figure 3.12

Debris slide along Arthur's Pass, South Island, New Zealand.

direct factor in causing rock slides, although it may weaken bonding along joints and bedding planes. Freeze–thaw action, however, is an important cause. Rock slides commonly occur on steep slopes, and most of them are of single rather than multiple occurrence. They are composed of rock boulders. Individual fragments may be very large and may move great distances from their source. Debris slides are usually restricted to the weathered zone or to surficial talus. With increasing water content, debris slides grade into mudflows. These slides are often limited by the contact between loose material and the underlying firm bedrock.

In a flow, the movement resembles that of a viscous fluid. Slip surfaces are usually not visible or are short lived, and the boundary between the flow and the material over which it moves may be sharp or may be represented by a zone of plastic flow. Some content of water is necessary for most types of flow movement, but dry flows can occur. Dry flows, which consist predominantly of rock fragments, are referred to as rock fragment flows or rock avalanches and generally result from a rock slide or rockfall turning into a flow. Generally, dry flows are very rapid and short lived, and frequently are composed mainly of silt or sand. As would be expected, they are of frequent occurrence in rugged mountainous regions where they usually involve the movement of many millions of tonnes of material. Wet flows occur when fine-grained soils, with or without coarse debris, become mobilized by an excess of water. They may be of great length.

Progressive failure is rapid in debris avalanches, and the whole mass, either because it is quite wet or is on a steep slope, moves downwards, often along a stream channel, and advances well beyond the foot of a slope. Debris avalanches are generally long and narrow, and frequently leave V-shaped scars tapering headwards. These gullies often become the sites of further movement.

Debris flows are distinguished from mudflows on the basis of particle size, the former containing a high percentage of coarse fragments, whereas the latter consist of at least 50% sand-size particles or less. Almost invariably, debris flows follow unusually intense rainfall or sudden thaw of frozen ground. These flows are of high density, perhaps 60 to 70% solids by weight, and are capable of carrying large boulders. Similar to debris avalanches, they commonly cut V-shaped channels, at the sides of which coarser material may accumulate as the more fluid central area moves down-channel. Both debris flows and mudflows may move over many kilometres.

Mudflows may develop when torrential rain or a rapidly moving stream of storm water mixes with a sufficient quantity of debris to form a pasty mass (Fig. 3.13). Because mudflows frequently occur along the same courses, they should be kept under observation when significant damage is likely to result. Mudflows frequently move at rates ranging between 10 and 100 m min^{-1} and can travel over slopes inclined at 1° or less. Indeed, they usually develop

Figure 3.13

Mudflow in colluvial ground, Durban, South Africa.

on slopes with shallow inclinations, that is, between 5 and 15°. An earthflow involves mostly cohesive or fine-grained material that may move slowly or rapidly. The speed of movement is, to some extent, dependent on water content in that the higher the content, the faster the movement. Slowly moving earthflows may continue to move for several years. These flows generally develop as a result of a build-up of pore water pressure, so that part of the weight of the material is supported by interstitial water with a consequent decrease in shearing resistance. A bulging frontal lobe is formed if the material is saturated, and this may split into a number of tongues that advance with a steady rolling motion. Earthflows frequently form the spreading toes of rotational slides due to the material being softened by the ingress of water.

Fluvial Processes

The Development of Drainage Systems

As far as the development of a drainage system is concerned, it is assumed that the initial drainage pattern develops on a new surface and consists of a series of sub-parallel rills flowing down the steepest slopes. The drainage pattern then becomes integrated by micropiracy (the beheading of the drainage system of a small rill by that of a larger one) and cross-grading. Micropiracy occurs when the ridges that separate the initial rills are overtopped and broken down. When the divides are overtopped, the water tends to move towards those rills

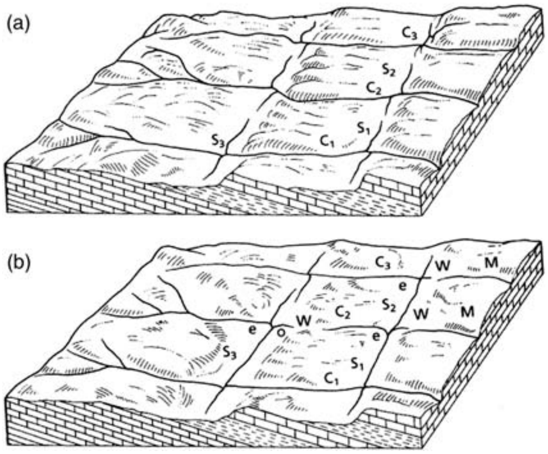


Figure 3.14

(a) Trellised drainage pattern of consequent streams, C, and their subsequents, S, showing the erosion of a gentle dipping series of hard and soft beds into escarpments. (b) Later development illustrating river capture or micropiracy by the headward growth of the more vigorous subsequent streams; e = elbow of capture, W = wind gap, M = misfit stream, o = obsequent stream.

at a slightly lower elevation and, in the process, the divides are eroded. Eventually, water drains from rills of higher elevation into adjacent ones of lower elevation (Fig. 3.14). The flow towards the master rill steadily increases, and its development across the main gradient is termed cross-grading. The tributaries that flow into the master stream are subsequently subjected to cross-grading and, thus the river system is gradually developed.

The texture of the drainage system is influenced by rock type and structure, the nature of the vegetation cover and the type of climate. The drainage density affords a measure of comparison between the development of one drainage system and another. It is calculated by dividing the total length of a stream by the area it drains, and is generally expressed in kilometres per square kilometre.

Streams can be classified into orders. First-order streams are unbranched, and when two such streams become confluent, they form a second-order stream. It is only when streams of the same order meet that they produce one of the higher rank, for example, a second-order stream flowing into a third-order stream does not alter its rank. The frequency with which streams of a certain order flow into those of the next order above them is referred to as the bifurcation ratio. The bifurcation ratio for any consecutive pair of orders is obtained by dividing the total number of streams of the lower order by the total number in the next higher order. Similarly, the stream length ratio is found by dividing the total length of streams of the lower order by the total length of those in the next higher order. Values of stream length ratio depend mainly on drainage density and stream entrance angles, and increase somewhat with increasing order. A river system also is assigned an order, which is defined numerically by the highest stream order it contains.

In the early stages of development, in particular, rivers tend to accommodate themselves to the local geology. For instance, tributaries may develop along fault zones. What is more, rock type has a strong influence on the drainage texture or channel spacing. In other words, a low drainage density tends to form on resistant or permeable rocks, whereas weak highly erodible rocks are characterized by a high drainage density.

The initial dominant action of master streams is vertical down-cutting that is accomplished by the formation of potholes, which ultimately coalesce, and by the abrasive action of the load. Hence, in the early stages of river development, the cross profile of the valley is sharply V-shaped. As time passes, valley widening due to soil creep, slippage, rain-wash and gullying becomes progressively more important and, eventually, lateral corrasion replaces vertical erosion as the dominant process. A river possesses few tributaries in the early stages, but their numbers increase as the valley widens, thus affording a growing increment of rock waste to the master stream, thereby enhancing its corrasive power.

During valley widening, the stream erodes the valley sides by causing undermining and slumping to occur on the outer concave curves of meanders where steep cliffs or bluffs are formed. These are most marked on the upstream side of each spur. Deposition usually takes place on the convex side of a meander. Meanders migrate both laterally and downstream, and their amplitude is increased progressively. In this manner, spurs are eroded continually, first becoming more asymmetrical until they are eventually truncated (Fig. 3.15). The slow deposition that occurs on the convex side of a meander, as lateral migration proceeds, produces a gently sloping area of alluvial ground called the flood plain. The flood plain gradually grows wider as the river bluffs recede, until it is as broad as the amplitude of the meanders. From now onwards, the continual migration of meanders slowly reduces the valley floor to an almost flat plain that slopes gently downstream and is bounded by shallow valley sides.

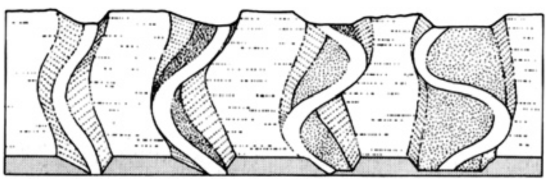


Figure 3.15

Widening of valley floor by lateral corrosion.

Throughout its length, a river channel has to adjust to several factors that change independently of the channel itself. These include the different rock types and structures across which it flows. The tributaries and inflow of water from underground sources affect the long profile of a river. Other factors that bring about adjustment of a river channel are flow resistance, which is a function of particle size, and the shape of transistory deposits such as bars, the method of load transport and the channel pattern including meanders and islands. Lastly, the river channel must also adjust itself to the river slope, width, depth and velocity.

As the longitudinal profile or thalweg of a river is developed, the differences between the upper, middle and lower sections of its course become more clearly defined until three distinctive tracts are observed. These are the upper or torrent, the middle or valley, and the lower or plain stages. The torrent stage includes the headstreams of a river where small fast-flowing streams are engaged principally in active downward and headward erosion. They possess steep-sided cross profiles and irregular thalwegs. The initial longitudinal profile of a river reflects the irregularities that occur in its path. For instance, it may exhibit waterfalls or rapids where it flows across resistant rocks. However, such features are transient in the life of a river. In the valley tract, the predominant activity is lateral corrasion. The shape of the valley sides depend on the nature of the rocks being excavated, the type of climate, the rate of rock wastage and meander development. Some reaches in the valley tract may approximate to grade, and the meanders may have developed alluvial flats there, while other stretches may be steep-sided with irregular longitudinal profiles. The plain tract is formed by the migration of meanders, and deposition is the principal river activity.

Meanders, although not confined to, are characteristic of flood plains. The consolidated veneer of alluvium, spread over a flood plain, offers little resistance to continual meander development. Hence, the loops become more and more accentuated. As time proceeds, the swelling loops approach one another. During flood, the river may cut through the neck, separating two adjacent loops, thereby straightening its course. As it is much easier for the river to flow through this new course, the meander loop is silted off and abandoned as an oxbow lake (Fig. 3.16).

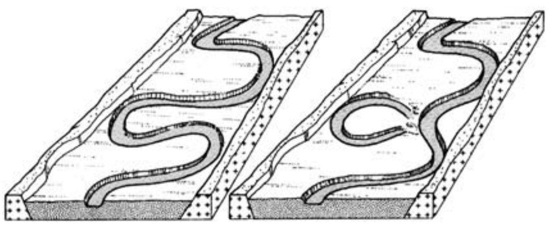


Figure 3.16

Formation of an oxbow lake.

Meander lengths vary from 7 to 10 times the width of the channel, whereas cross-overs occur at about every 5 to 7 channel widths. The amplitude of a meander bears little relation to its length but is largely determined by the erosion characteristics of the river bed and local factors. For instance, in uniform material, the amplitude of meanders does not increase progressively nor do meanders form oxbow lakes during the downstream migration of bends. Higher sinuosity is associated with small width relative to depth and a larger percentage of silt and clay in the river banks, which affords greater cohesiveness. Relatively sinuous channels with a low width–depth ratio are developed by rivers transporting large quantities of suspended sediment. By contrast, the channel tends to be wide, shallow and less sinuous when the amount of bedload discharge is high.

A river is described as being braided if it splits into a number of separate channels or anabranches to adjust to a broad valley. The areas between the anabranches are occupied by islands built of gravel and sand. For the islands to remain stable, the river banks must be more erodible so that they give way rather than the islands. Braided channels occur on steeper slopes than do meanders.

Climatic changes and earth movements alter the base level to which a river grades. When a land surface is elevated, the down-cutting activity of rivers flowing over it is accelerated. The rivers begin to regrade their courses from their base level and, as time proceeds, their newly graded profiles are extended upstream until they are fully adjusted to the new conditions. Until this time, the old longitudinal profile intersects with the new to form a knick point. The upstream migration of knick points tends to be retarded by outcrops of resistant rock; consequently, after an interval of time they are usually located at hard rock exposures. The acceleration of down-cutting consequent on uplift frequently produces a new valley within the old, the new valley extending upstream to the knick point.

River terraces are also developed by rejuvenation. In the lower course of a river, uplift leads to the river cutting into its alluvial plain. The lateral and downstream migration of meanders means that a new flood plain is formed but very often paired alluvial terraces, representing the remnants of the former flood plain, are left at its sides (Fig. 3.17).

Incised meanders are also associated with rejuvenation and are often found together with river terraces. When uplift occurs, the down-cutting action of meanders is accelerated, and they carve themselves into the terrain over which they flow. The landforms that are then produced depend on the character of the terrain, and the relative rates of down-cutting and meander migration. If vertical erosion is rapid, meander shift has little opportunity to develop and, consequently, the loops are not greatly enlarged. The resulting incised meanders are described as entrenched. However, when time is afforded for meander migration, they incise themselves by oblique erosion and the loops are enlarged, then they are referred to as ingrown meanders.

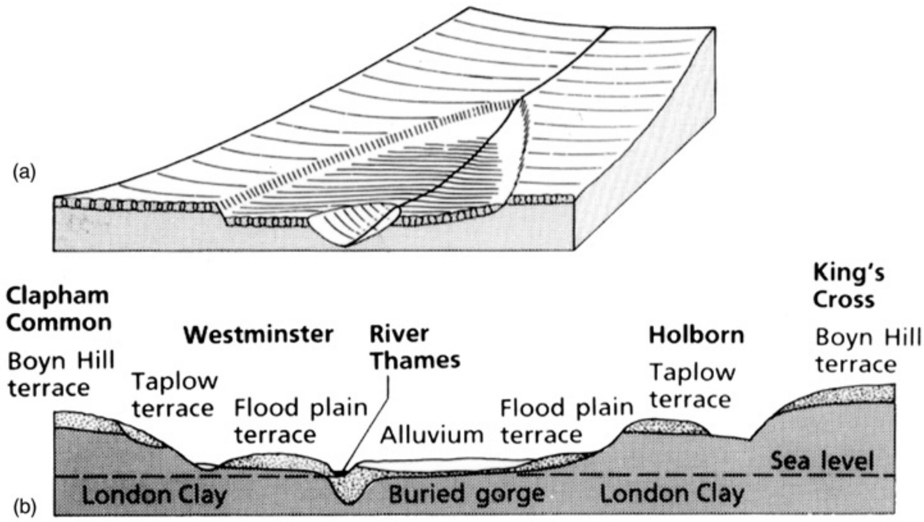


Figure 3.17

(a) Paired river terraces due to rejuvenation, note valley in valley and knick points. (b) Section across London, England, to show the paired terraces and one of the buried valleys of the River Thames.

When incision occurs in the alluvium of a river plain, the meanders migrate back and forth across the floor. On each successive occasion that a meander swings back to the same side, it does so at a lower level. As a consequence, small remnant terraces may be left above the newly formed plain. These terraces are not paired across the valley, and their position and preservation depends on the swing of meanders over the valley. If down-cutting is very slow, then erosion terraces are unlikely to be preserved.

Conversely, when sea level changes, rivers again have to regrade their courses to the new base level. For instance, during glacial times, the sea level was at a much lower level, and rivers carved out valleys accordingly. As the last glaciation retreated, the sea level rose and the rivers had to adjust to these changing conditions. This frequently meant that their valleys were filled with sediments. Hence, buried channels are associated with the lowland sections of many rivers.

The Work of Rivers

The work done by a stream is a function of the energy it possesses. Stream energy is lost as a result of friction from turbulent mixing, and frictional losses are dependent on channel roughness and shape. Total energy is influenced mostly by velocity, which is a function of the stream gradient, volume and viscosity of water in flow and the characteristics of the channel cross section and bed. This relationship has been embodied in the Chezy formula, which expresses velocity as a function of hydraulic radius, *R*, and slope, *S*:

$$v = C\sqrt{(RS)} \tag{3.7}$$

where *v* is mean velocity and *C* is a constant that depends on gravity and varies with the characteristics of the channel. Numerous attempts have been made to find a generally acceptable expression for *C*.

The Manning formula represents an attempt to refine the Chezy equation in terms of the constant *C*:

$$v = \frac{1.49}{n} R^{2/3}\ S^{1/2} \tag{3.8}$$

where the terms are the same as in the Chezy equation, and *n* is the roughness factor. The velocity of flow increases as roughness decreases for a channel of particular gradient and dimensions. The roughness factor has to be determined empirically and varies not only for different streams but also for the same stream under different conditions and at different times. In natural channels, the value of *n* is 0.01 for smooth beds, about 0.02 for sand and 0.03 for gravel. Anything that affects the roughness of a channel changes *n*, including the size and shape of grains on the bed, sinuosity and obstructions in the channel section. Variation in discharge also affects the roughness factor since depth of water and volume influence the roughness.

The ratio between the cross-sectional area of a river channel and the length of its wetted perimeter determines the efficiency of the channel. This ratio is termed the hydraulic radius, and the higher its value, the more efficient is the river. The most efficient forms of channel are those with approximately circular or rectangular sections, with widths approaching twice their depths. On the other hand, the most inefficient channel forms are very broad and shallow with wide wetted perimeters.

The quantity of flow can be estimated from measurements of cross-sectional areas and current speed of a river. Generally, channels become wider relative to their depth and adjusted to larger flows with increasing distance downstream. Bankfull discharges also increase downstream in proportion to the square of the width of the channel or of the length of individual meanders, and in proportion to the 0.75 power of the total drainage area focused at the point in question.

Statistical methods are used to predict river flow and assume that recurrence intervals of extreme events bear a consistent relationship to their magnitudes. A recurrence interval,

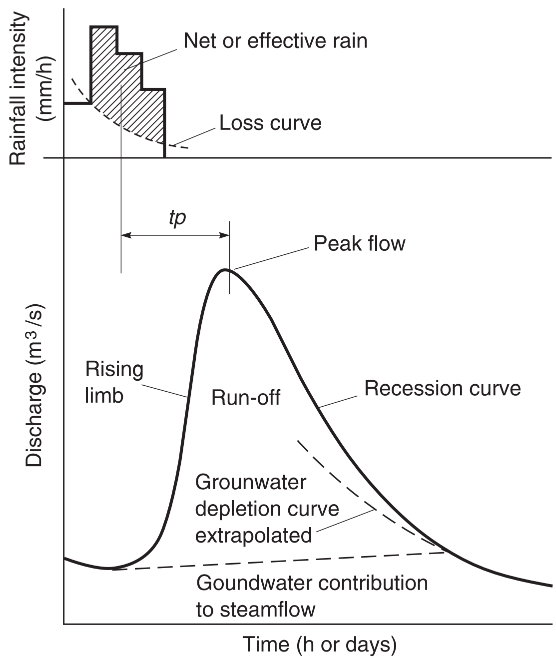


Figure 3.18

Component parts of a hydrograph. When rainfall commences, there is an initial period of interception and infiltration before any measurable runoff reaches the stream channels. During the period of rainfall, these losses continue in a reduced form so that the rainfall graph has to be adjusted to show effective rain. When the initial losses are met, surface runoff begins and continues to a peak value that occurs at time *tp*, measured from the centre of gravity of the effective rain on the graph. Thereafter, surface runoff declines along the recession limb until it disappears. Baseflow represents the groundwater contribution along the banks of the river.

generally of 50 or 100 years, is chosen in accordance with the given hydrological requirements. The concept of unit hydrograph postulates that the most important hydrological characteristics of any basin can be seen from the direct run-off hydrograph resulting from 25 mm of rainfall evenly distributed over 24 h. This is produced by drawing a graph of the total stream flow at a chosen point as it changes with time after such a storm, from which the normal baseflow caused by groundwater is subtracted (Fig. 3.18).

There is a highly significant relationship between mean annual flood discharge per unit area and drainage density. Peak discharge and the lag time of discharge (the time that elapses between maximum precipitation and maximum run-off) are also influenced by drainage density, as well as by the shape and slope of the drainage basin. Stream flow is generally most variable, and flood discharges at a maximum per unit area in small basins. This is because storms tend to be local in occurrence. A relationship exists between drainage density and baseflow or groundwater discharge. This is related to the permeability of the rocks present in

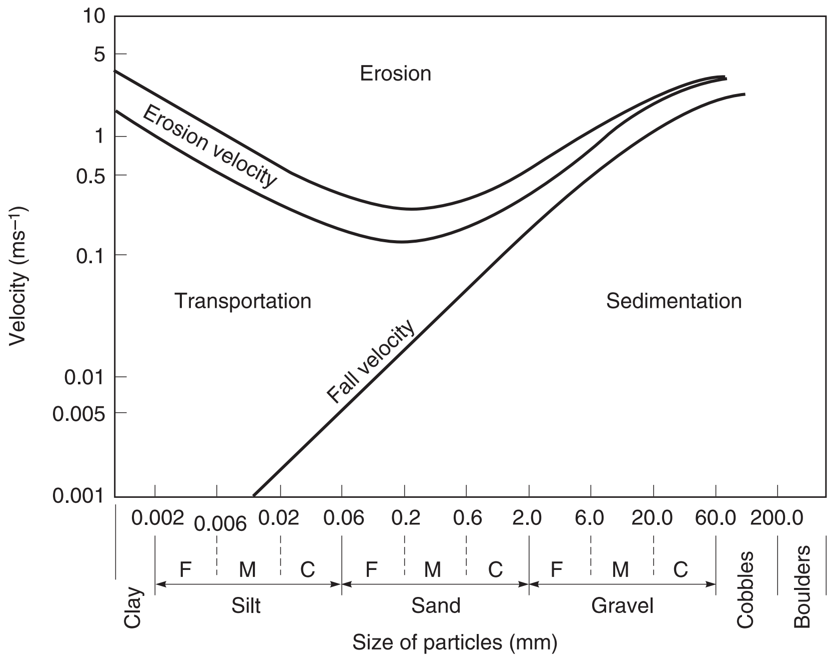


Figure 3.19

Curves for erosion, transportation and deposition of uniform sediment. F = fine; M = medium; C = coarse. Note that sand is the most easily eroded material.

a drainage basin. In other words, the greater the quantity of water that moves on the surface of the drainage system, the higher the drainage density, which, in turn, means that the base-flow is lower. As pointed out previously, in areas of high drainage density, the soils and rocks are relatively impermeable and water runs off rapidly. The amount of infiltration is reduced accordingly.

The work undertaken by a river is threefold, it erodes soils and rocks and transports the products thereof, which it eventually deposits (Fig. 3.19). Erosion occurs when the force provided by the river flow exceeds the resistance of the material over which it runs. The velocity needed to initiate movement, that is, the erosional velocity, is appreciably higher than that required to maintain movement. Four types of fluvial erosion have been distinguished, namely, hydraulic action, attrition, corrasion and corrosion. Hydraulic action is the force of the water itself. Attrition is the disintegration that occurs when two or more particles that are suspended in water collide. Corrasion is the abrasive action of the load carried by a river on its channel. Most of the erosion done by a river is attributable to corrasive action. Hence, a river carrying coarse, resistant, angular rock debris possesses a greater ability to erode than does one transporting fine particles in suspension. Corrosion is the solvent action of river water.

In the early stages of river development, erosion tends to be greatest in the lower part of the drainage basin. However, as the basin develops, the zone of maximum erosion moves upstream, and it is concentrated along the divides in the later stages. The amount of erosion accomplished by a river in a given time depends on its volume and velocity of flow; the character and size of its load; the rock type and geological structure over which it flows; the infiltration capacity of the area it drains; the vegetation (which affects soil stability); and the permeability of the soil. The volume and velocity of a river influence the quantity of energy it possesses. When flooding occurs, the volume of a river is increased significantly, which leads to an increase in its velocity and competence. However, much energy is spent in overcoming the friction between the river and its channel so that energy losses increase with any increase in channel roughness. Obstructions, changing forms on a river bed such as sandbars and vegetation, offer added resistance to flow. Bends in a river also increase friction. Each of these factors causes deflection of the flow that dissipates energy by creating eddies, secondary circulation and increased shear rate.

The load that a river carries is transported in four different ways. Firstly, there is traction, that is, rolling of the coarsest fragments along the river bed. Secondly, smaller material, when caught in turbulent upward-moving eddies, proceeds downstream in a jumping motion referred to as saltation. Thirdly, fine sand, silt and mud may be transported in suspension. Fourthly, soluble material is carried in solution.

Sediment yield may be determined by sampling both the suspended load and the bedload. It can also be derived from the amount of deposition that takes place when a river enters a relatively still body of water such as a lake or a reservoir.

The competence of a river to transport its load is demonstrated by the largest boulder it is capable of moving; it varies according to the velocity of a river and its volume, being at a maximum during flood. It has been calculated that the competence of a river varies as the sixth power of its velocity. The capacity of a river refers to the total amount of sediment that it carries. It varies according to the size of the soil and rock material that form the load, and the velocity of the river. When the load consists of fine particles, the capacity is greater than when it is comprised of coarse material. Usually, the capacity of a river varies as the third power of its velocity.

Both the competence and capacity of a river are influenced by changes in the weather, and the lithology and structure of the rocks over which it flows, as well as by vegetative cover and land use. Because the discharge of a river varies, all sediments are not transported continuously, for instance, boulders may be moved only a few metres during a single flood. Sediments that are deposited over a flood plain may be regarded as being stored there temporarily.

Deposition occurs where turbulence is at a minimum or where the region of turbulence is near the surface of a river. For example, lateral accretion occurs, with deposition of a point bar, on the inside of a meander bend. The settling velocity for small grains in water is roughly proportional to the square of the grain diameter, whereas for larger particles, settling velocity is proportional to the square root of the grain diameter. An individual point bar grows as a meander migrates downstream, and new ones are formed as a river changes its course during flood. Indeed, old meander scars are a common feature of flood plains. The combination of point bar and filled slough or oxbow lake gives rise to ridge and swale topography. The ridges consist of sandbars, and the swales are sloughs filled with silt and clay.

An alluvial flood plain is the most common depositional feature of a river. The alluvium is made up of many kinds of deposits, laid down both in the channel and outside it. Vertical accretion on a flood plain is accomplished by in-channel filling and the growth of overbank deposits during and immediately after floods. Gravel and coarse sands are moved chiefly at flood stages and deposited in the deeper parts of a river. As the river overtops its banks, its ability to transport material is lessened, so that coarser particles are deposited near the banks to form levees. Levees stand above the general level of the adjoining plain so that the latter is usually poorly drained and marshy (Fig. 3.20). This is particularly the case when levees have formed across the confluences of minor tributaries, forcing them to wander over the flood plain until they find another entrance to the main river. Finer material is carried farther and laid down as backswamp deposits. At this point, a river sometimes aggrades its bed, eventually raising it above the level of the surrounding plain. Consequently, when levees are breached by flood water, hundreds of square kilometres may be inundated.

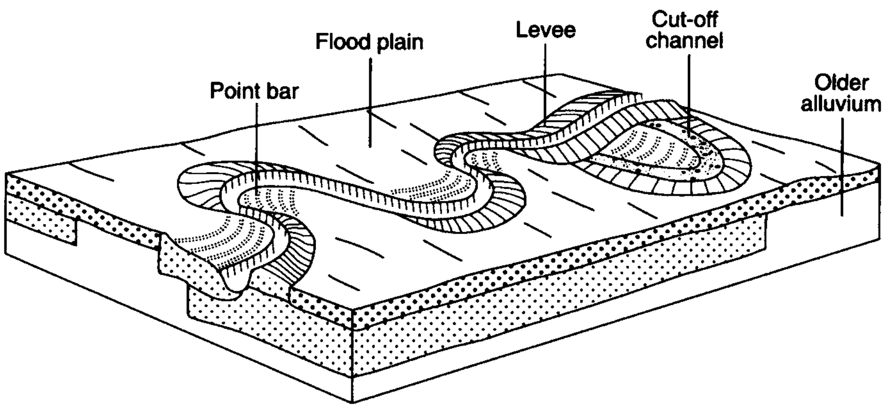


Figure 3.20

The main depositional features of a meandering channel.

Karst Topography and Underground Drainage

Karst topography refers to a distinctive terrain that is associated primarily with carbonate rock masses, which contain physical features that have been formed by dissolution of the rock material. These features include sinkholes, dry valleys, pavements and subsurface drainage and associated springs, voids, galleries and caves. Karst terrain, therefore, not only possesses features peculiar to itself but also unique hydrogeological characteristics. The features vary enormously in character, shape and size, and so may represent extremely difficult and costly ground conditions to work in. Although karst is associated primarily with limestone and dolostone rock masses, karstic features are developed in chalk and marble, as well as in evaporitic rocks, especially gypsum.

Suites of karst landforms in limestone and related carbonate rocks evolve through progressive denudation of the land surface, while underground denudation is simultaneously enlarging cave conduits so that ever larger proportions of the drainage can pass underground. Both surface and underground denudation is largely by dissolution of the carbonate, at rates dependent on the flow and chemical aggressivity of the water. Both these factors are dependent on climate. If solution continues, its rate slackens and it eventually ceases when saturation is reached. Therefore, solution is greatest when the bicarbonate saturation is low. This occurs when water is circulating so that fresh supplies with low bicarbonate saturation are made available continually. Water flows are largely a consequence of rainfall input (though they can be increased locally by supplies of allogenic water draining off adjacent outcrops of non-karstic rocks).

The dissolution of limestone is a very slow process. For instance, Kennard and Knill (1968) quoted mean rates of surface lowering of limestone areas in the British Isles that ranged from 0.041 to 0.099 mm annually, and Sowers (1996) suggested rates of 0.025 to 0.040 mm a^{-1} for the eastern United States. More recently, Trudgill and Viles (1998) quoted calculated erosion rates of calcite of 0.06 to 0.11 mm a^{-1} at pH 5.5, and 2.18 to 2.69 mm a^{-1} at pH 4.0. Nevertheless, solution may be accelerated by man-made changes in the groundwater conditions or by a change in the character of the surface water that drains into limestone.

Limestone and dolostone are transected by discontinuities. These normally have been subjected to various degrees of dissolution so that they gape. The progressive opening of discontinuities by dissolution leads to an increase in rock mass permeability. Moreover, dissolution along discontinuities produces an irregular surface that is characterized by the presence of clints and grykes (i.e. slabs of limestone separated by furrows) as, for example, may be seen on limestone pavements. The latter are bare limestone surfaces that are associated with near-horizontally bedded limestone (Fig. 3.21). Rockhead profiles developed beneath soil profiles may also be irregular as again discontinuities, notably joints, are subjected to dissolution. In both cases, this process ultimately leads to the formation of rock pinnacles of various sizes.

Figure 3.21

Limestone pavement, the Burren, Southern Ireland.

Sinkholes are characteristic features of karst terrain and may develop where solution-opened joints intersect (Waltham and Fookes, 2003). They may lead to an integrated system of subterranean galleries and caves. The latter are characteristic of thick massive limestone or dolostone formations. Sinkholes vary in diameter but usually are a few metres or tens of metres across and may descend up to 500 m in depth below the surface (Fig. 3.22).

Caves form in competent limestone or dolostone where there is an adequate through flow of groundwater, the flow rate and aggressiveness of groundwater being mainly responsible for the enlargement of caves. They originate along bedding planes and tectonic fractures, which are enlarged into networks of open fissures, favourable flow paths being enlarged selectively into caves. It is frequently asserted that the most active dissolution occurs at and just below the water table (Trudgill and Viles, 1998). Hence, it is here, within the uppermost part of the phreatic zone, where caves are most likely to be developed. As such, they are often within 100 m of the ground surface. If the amplitude of fluctuation of the water table increases or if it suffers a notable decline, this can lead to the enlargement of caves. Subsequently, caves may be abandoned if their groundwater is captured by preferred routes or they may be partially or wholly filled with sediment. The size of caves

Figure 3.22

Appearance of a sinkhole, Guilin, China.

ranges up to huge caverns, the largest being found in the humid tropics. As an example, Swart et al. (2003) reported that the Apocalypse cave system, which occurs in the dolostone of the Wonderfontein Valley, South Africa, has a surveyed length of approximately 13 km and is considered the longest cave system in southern Africa. A cave collapses when its span exceeds its bridging capacity, which depends primarily on rock mass strength (which takes character of discontinuities and degree of weathering into account), and in doing so, the void migrates upward. Nevertheless, most caves in massive strong limestone/dolostone are stable and are generally located at depths at which stable roof arches have developed.

Surface streams disappear underground via sinkholes. The larger sinkholes may be connected near the surface by irregular inclined shafts, known as ponors, to integrated underground systems of galleries and caves. Larger surface depressions form when enlarged sinkholes coalesce to form uvalas. These features may range up to 1 km in diameter. Any residual masses of limestone that, after a lengthy period of continuous denudation, remain as isolated hills are known as hums. These are honeycombed with galleries, shafts and caves. However, the nature of karst landforms is influenced by climate, for example, tower karst is developed in tropical and semi-tropical regions (Fig. 3.23).

Figure 3.23

Limestone towers, south of Guilin, China.

Underground systems of drainage in limestone terrains deepen and widen their courses by mechanical erosion and solution. During flood, water may wear away the roofs as well as the sides and floors of the galleries and caves through which it flows. In this way, caves are enlarged and their roofs are gradually thinned until they become unstable. At this point, they may partially or wholly collapse to form natural arches or gorges, respectively. Nevertheless, this is not a common occurrence.

Surface drainage is usually sparse in areas of thick limestone. Dry valleys are common, although they may be occupied by streams during periods of intense rainfall. Underground streams may appear as vaclusian springs where the water table meets the surface. Occasionally, streams that rise on impervious strata may traverse a broad limestone outcrop without disappearing.

Glaciation

A glacier may be defined as a mass of ice that is formed from recrystallized snow and refrozen melt water, and that moves under the influence of gravity. Glaciers develop above the snowline, that is, in regions of the world that are cold enough to allow snow to remain on the surface throughout the year. The snowline varies in altitude from sea level in polar regions to above 5000 m in equatorial regions. As the area of a glacier that is exposed to wastage is small compared with its volume, this accounts for the fact that glaciers penetrate into the warmer zones below the snowline.

Glaciers can be grouped into three types, namely, valley glaciers, piedmont glaciers, and ice sheets and ice caps. Valley glaciers flow down pre-existing valleys from mountains where snow has collected and formed into ice. They disappear where the rate of melting exceeds the rate of supply of ice. When a number of valley glaciers emerge from a mountain region onto a plain, where they coalesce, they then form a piedmont glacier. Ice sheets are huge masses of ice that extend over areas that may be of continental size; ice caps are of smaller dimensions. At present, there are two ice sheets in the world, one extends over the continent of Antarctica, whereas the other covers most of Greenland.

Glacial Erosion

Although pure ice is a comparatively ineffective agent as far as eroding massive rocks is concerned, it does acquire rock debris, which enhances its abrasive power. The larger fragments of rock embedded in the sole of a glacier tend to carve grooves in the path over which it travels, whereas the finer material smoothes and polishes rock surfaces. Ice also erodes by a quarrying process, whereby fragments are plucked from rock surfaces. Generally, quarrying is a more effective form of glacial erosion than is abrasion.

The rate of glacial erosion is extremely variable and depends on the velocity of the glacier, the weight of the ice, the abundance and physical character of the rock debris carried at the bottom of the glacier and the resistance offered by the rocks of the glacier channel. The erodibility of the surface over which a glacier travels varies with depth and, hence, with time. Once the weathered overburden and open-jointed bedrock have been removed, the rate of glacial erosion slackens. This is because quarrying becomes less effective and, hence, the quantity of rock fragments contributed to abrasive action is gradually reduced.

Continental ice sheets move very slowly and may be effective agents of erosion only temporarily, removing the weathered mantle from, and smoothing off the irregularities of, a landscape. The pre-glacial relief features are consequently afforded some protection by the overlying ice against denudation, although the surface is modified somewhat by the formation of hollows and hummocks.

The commonest features produced by glacial abrasion are striations on rock surfaces that were formed by rock fragments embedded in the base of the glacier. Many glaciated slopes formed of resistant rocks that are well jointed display evidence of erosion in the form of ice-moulded

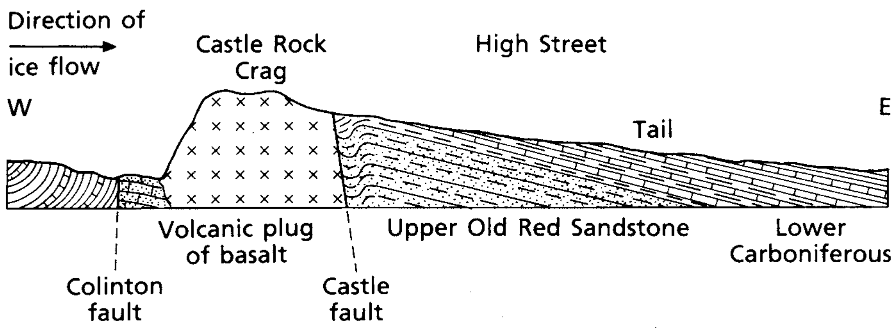


Figure 3.24

Crag and tail, Castle Rock, Edinburgh, Scotland. Castle Rock probably represents an early phase of volcanic activity associated with the volcano at Arthur's Seat (Carboniferous).

hummocks that are known as roche moutonnées. Large, highly resistant obstructions, such as volcanic plugs, which lie in the path of advancing ice, give rise to features called crag and tail (Fig. 3.24). The resistant obstruction forms the crag and offers protection to the softer rocks that occur on its leeward side.

Drumlins are mounds that are rather similar in shape to the inverted bowl of a spoon (Fig. 3.25). They vary in composition, ranging from 100% bedrock to 100% glacial deposit. Obviously, those types formed of bedrock originated as a consequence of glacial erosion, however, even those composed of glacial debris were, at least in part, moulded by glacial action. Drumlins range up to a kilometre in length, and some may be over 70 m in height. Usually, they do not occur singly but in scores or even hundreds in drumlin fields.

Corries are located at the head of glaciated valleys, being the features in which ice accumulated. Accordingly, they formed at the snowline, or close by. Corrie stairways are frequently arranged in tiers up a mountain side. Because of their shape, corries are often likened to amphitheatres in that they are characterized by steep backwalls and steep sides (Fig. 3.26). Their floors are generally rock basins. Corries vary in size, some of the largest being about 1 km across. The dominant factor influencing their size is the nature of the rock in which they were excavated.

The cross-profile of a glaciated valley is typically steep sided with a comparatively broad, flat bottom, and it is commonly referred to as U-shaped (Fig. 3.27). Most glaciated valleys are straighter than those of rivers because their spurs have been truncated by ice. In some glaciated valleys, a pronounced bench or shoulder occurs above the steep walls of the trough. Tributary streams of ice flow across the shoulders to the main glacier. When the ice disappears, the tributary valleys are left hanging above the level of the trough floor. The valleys are

Figure 3.25

Drumlins, near Downpatrick, Northern Ireland.

Figure 3.26

Corrie near Athabasca Glacier, Alberta, Canada.

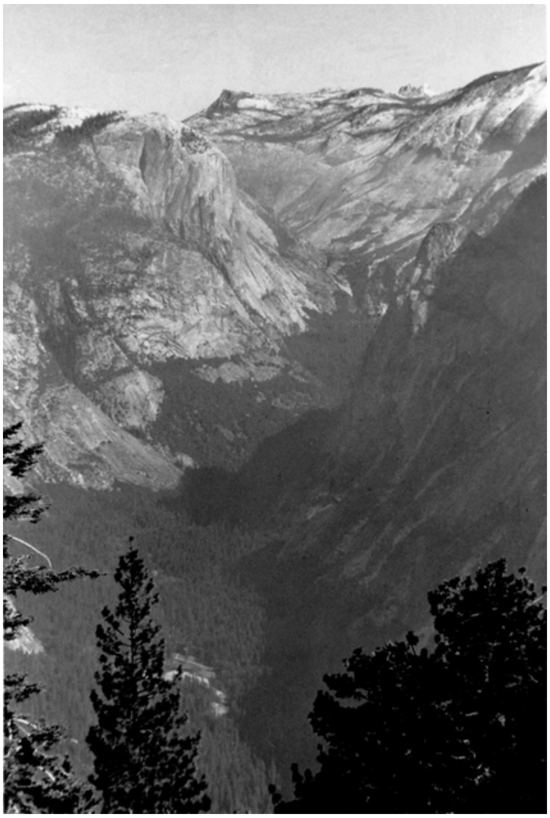


Figure 3.27

A stepped "U"-shaped valley, Yosemke National Park, California.

then occupied by streams. Those in the hanging valleys cascade down the slopes of the main trough as waterfalls. An alluvial cone may be deposited at the base of the waterfall.

Generally, glaciated valleys have a scalloped or stepped long profile, and sometimes the head of the valley is terminated by a major rock step known as a trough end. Such rock steps develop where a number of tributary glaciers, descending from corries at the head of the valley, converge and thereby effectively increase the erosive power. A simple explanation of a scalloped valley floor can be found in the character of the rock type. Not only is a glaciated valley stepped, but reversed gradients are also encountered within its path. The reversed gradients are located in rock-floored basins that occur along the valley. Rock basins appear to be formed by localized ice action.

Fiords are found along the coasts of glaciated highland regions that have suffered recent submergence; they represent the drowned part of a glaciated valley. Frequently, a terminal rock barrier, the threshold, occurs near the entrance of a fiord. Some thresholds rise very close to sea level, indeed some may be uncovered at low tide. However, water landward of the thresholds is very often deeper than the known post-glacial rise in sea level. For example, depths in excess of 1200 m have been recorded in some Norwegian fiords.

Glacial Deposits: Unstratified Drift

Glacial deposits form a more significant element of the landscape in lowland areas than they do in highlands. Two kinds of glacial deposits are distinguished, namely, unstratified drift or till and stratified drift. However, one type commonly grades into the other. Till is usually regarded as being synonymous with boulder clay and is deposited directly by ice, while stratified drift is deposited by melt waters or in proglacial lakes.

Till consists of a variable assortment of rock debris that ranges in size from fine rock flour to boulders and is characteristically unsorted (Bell, 2002). The compactness of a till varies according to the degree of consolidation undergone, the amount of cementation and size of the grains. Tills that contain less than 10% clay fraction are usually friable, whereas those with over 10% clay tend to be massive and compact.

Distinction has been made between tills derived from rock debris carried along at the base of a glacier and those deposits that were transported within or at the terminus of the ice. The former is sometimes referred to as lodgement till, whereas the latter is termed ablation till. Lodgement till is commonly compact and fissile, and the fragments of rock it contains are frequently orientated in the path of ice movement. Ablation till accumulates as the ice, in which the parent material is entombed, melts. Hence, it is usually uncompacted and non-fissile, and the boulders present display no particular orientation. Since ablation till consists only of the load carried at the time of ablation, it usually forms a thinner deposit than does a lodgement till.

A moraine is an accumulation of material deposited directly from a glacier. There are six types of moraines deposited by valley glaciers. Rock debris that a glacier wears from its valley sides and which is supplemented by material falling from the valley slopes above the ice forms the lateral moraine. When two glaciers become confluent, a medial moraine develops from the merger of the two inner lateral moraines. Material that falls onto the surface of a glacier and then makes its way via crevasses into the centre, where it becomes entombed, is termed englacial moraine. Some of this debris, however, eventually reaches the base of the glacier and enhances the material eroded from the valley floor. This constitutes the subglacial moraine. The ground moraine is often distributed irregularly since it is formed when basal ice becomes overloaded with rock debris and is forced to deposit some of it. The material that is deposited at the snout of a glacier when the rate of wastage is balanced by the rate of outward flow of ice is known as a terminal moraine (Fig. 3.28). Terminal moraines possess

Figure 3.28

Terminal moraine, Tasman Glacier, South Island, New Zealand.

a curved outline impressed upon them by the lobate nature of the snout of the ice. They are usually discontinuous, being interrupted where streams of melt water issue from the glacier. Frequently, a series of terminal moraines may be found traversing a valley, the farthest down-valley marking the point of maximum extension of the ice, the others indicating pauses in glacial retreat. The latter types are called recessional moraines.

Ground moraines and terminal moraines are the two principal types of moraines deposited by ice sheets that spread over lowland areas. In lowland areas, the terminal moraines of ice sheets may rise to heights of some 60 m. In plan, they commonly form a series of crescents, each crescent corresponding to a lobe at the snout of the ice. If copious amounts of melt water drain from the ice front, then morainic material is washed away, and hence a terminal moraine either does not develop, or if it does, is of inconspicuous dimension.

Fluvio-Glacial Deposits; Stratified Drift

Stratified deposits of drift often are subdivided into two categories, namely, those deposits that accumulate beyond the limits of the ice, forming in streams, lakes or seas, and those deposits that develop in contact with the ice. The former types are referred to as pro-glacial deposits, and the latter are termed ice-contact deposits.

Most melt water streams that deposit outwash fans do not originate at the snout of a glacier but from within or upon the ice. Many of the streams that flow through a glacier have steep gradients and are, therefore, efficient transporting agents, but when they emerge at the snout, they do so on to a shallower incline, and deposition results. Outwash deposits typically are cross-bedded and range in size from boulders to coarse sand. When first deposited, the porosity of these sediments varies from 25 to 50%. Therefore, they are very permeable and so can resist erosion by local run-off. The finer silt–clay fraction is transported further downstream. Also, in this direction, an increasing amount of stream alluvium is contributed by tributaries, so that eventually the fluvio-glacial deposits cannot be distinguished. Many outwash masses are terraced.

Five different types of stratified drift deposited in glacial lakes have been recognized, namely, terminal moraines, deltas, bottom deposits, ice-rafted erratics and beach deposits. Terminal moraines that formed in glacial lakes differ from those that arose on land in that lacustrine deposits are inter-stratified with drift. Glacial lake deltas are usually composed of sands and gravels that are typically cross-bedded. By contrast, those sediments that accumulated on the floors of glacial lakes are fine-grained, consisting of silts and clays. These fine-grained sediments are sometimes composed of alternating laminae of finer and coarser grain size. Each couplet has been termed a varve. Large boulders that occur on the floors of glacial lakes were transported on rafts of ice and were deposited when the ice melted. Usually, the larger the glacial lake, the larger were the beach deposits that developed about it. If changes in lake level took place, then these may be represented by a terraced series of beach deposits.

Deposition that takes place at the contact of a body of ice is frequently sporadic and irregular. Locally, the sediments possess a wide range of grain size, shape and sorting. Most are granular, and variations in their engineering properties reflect differences in particle size distribution and shape. Deposits often display abrupt changes in lithology and, consequently, in relative density. They are deformed since they sag, slump or collapse as the ice supporting them melts.

Kame terraces are deposited by melt water streams that flow along the contact between the ice and the valley side (Fig. 3.29). The drift is derived principally from the glacier, although some is supplied by tributary streams. They occur in pairs, one on each side of the valley. If a series of kame terraces occur on the valley slopes, then each pair represents a pause in the process of glacier thinning. The surfaces of these terraces are often pitted with kettle holes (depressions where large blocks of ice remained unmelted while material accumulated around them). Narrow kame terraces are usually discontinuous, spurs having impeded deposition.

Kames are mounds of stratified drift that originate as small deltas or fans built against the snout of a glacier where a tunnel in the ice, along which melt water travels, emerges (Fig. 3.30).

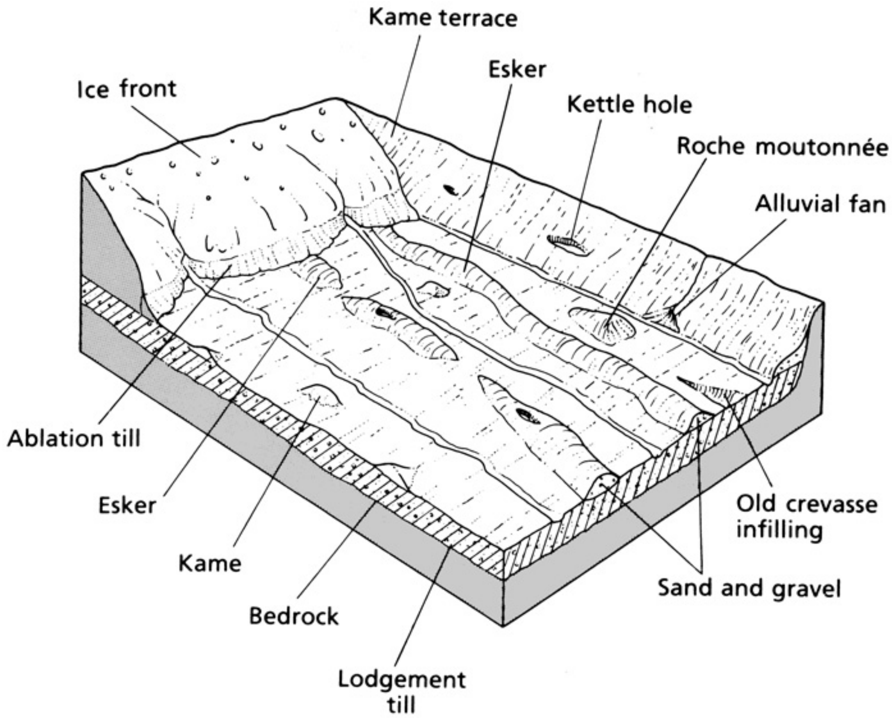


Figure 3.29

Block diagram of a glaciated valley showing typical glacial and fluvio-glacial deposits.

Figure 3.30

A kame being exploited for sand, north of Lillehammer, Norway.

Other small ridge-like kames accumulate in crevasses in stagnant or near-stagnant ice. Many kames do not survive deglaciation for any appreciable period of time.

Eskers are long, narrow, sinuous, ridge-like masses of stratified drift that are unrelated to surface topography (Fig. 3.29). For example, eskers may climb up valley sides and cross low watersheds. They represent sediments deposited by streams that flowed within channels in a glacier. Although eskers may be interrupted, their general continuity is easily discernible and, indeed, some may extend lengthwise for several hundred kilometres. Eskers may reach up to 50 m in height, and they range up to 200 m in width. Their sides are often steep. Eskers are composed principally of sands and gravels, although silts and boulders are found within them. These deposits are generally cross-bedded.

Other Glacial Effects

Ice sheets have caused diversions of drainage in areas of low relief. In some areas that were completely covered with glacial deposits, the post-glacial drainage pattern may bear no relationship to the surface beneath the drift, indeed moraines and eskers may form minor water divides. As would be expected, notable changes occurred at or near the margin of the ice. Lakes were formed there that were drained by streams whose paths disregarded pre-glacial relief. Evidence of the existence of pro-glacial lakes is to be found in the lacustrine deposits, terraces and overflow channels that they leave behind.

Where valley glaciers extend below the snowline, they frequently pond back streams that flow down the valley sides, giving rise to lakes. If any col between two valleys is lower than the surface of the glacier occupying one of them, then the water from any adjacent lake dammed by this glacier eventually spills into the adjoining valley, and in so doing, erodes an overflow channel. Marginal spillways may develop along the side of a valley at the contact with the ice.

The enormous weight of an overlying ice sheet causes the Earth’s crust beneath it to sag. Once the ice sheet disappears, the land slowly rises to recover its former position and, thereby, restores isostatic equilibrium. Consequently, the areas of northern Europe and North America presently affected by isostatic uplift more or less correspond with those areas that were formerly covered with ice. At present, the rate of isostatic recovery, for example, in the centre of Scandinavia, is approximately one metre per century. Isostatic uplift is neither regular nor continuous. Consequently, the rise in the land surface so affected has been overtaken at times by a rise in sea level. The latter was caused by melt water from the retreating ice sheets.

With the advance and retreat of ice sheets in Pleistocene times, the level of the sea fluctuated. Marine terraces (strandlines) were produced during interglacial periods when the sea was at a much higher level. The post-glacial rise in sea level has given rise to drowned coastlines such as rias and fiords, young developing cliffines, aggraded lower stretches of river valleys, buried channels, submerged forests, marshlands, shelf seas, straits and the reformation of numerous islands.

Frozen Ground Phenomena in Periglacial Environments

Frozen ground phenomena are found in regions that experience a tundra climate, that is, in those regions where the summer temperatures are only warm enough to cause thawing in the upper metre or so of the soil. Beneath the upper or active zone, the subsoil is permanently frozen and is hence known as the permafrost layer. Because of this layer, summer melt water cannot seep into the ground, the active zone becomes waterlogged and the soils on gentle slopes are liable to flow. Layers or lenses of unfrozen ground termed taliks may occur, often temporarily, in the permafrost (Fig. 3.31).

Permafrost is an important characteristic, although it is not essential to the definition of periglacial conditions, the latter referring to conditions under which frost action is the predominant weathering agent. Permafrost covers 20% of the land surface of the Earth and, during Pleistocene times, it was developed over an even larger area. Ground cover, surface water, topography and surface materials all influence the distribution of permafrost. The temperature

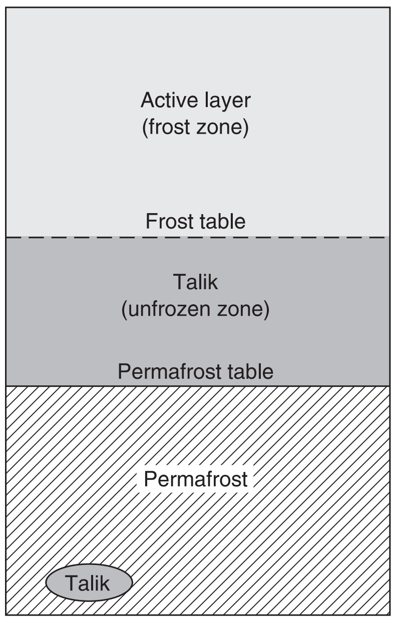


Figure 3.31

Terminology of some features associated with permafrost.

of perennially frozen ground below the depth of seasonal change ranges from slightly less than 0 to –12°C. Generally, the higher the latitude, the lesser the depth of thaw. It is at a minimum in peat or highly organic sediments and increases in clay, silt and sand to a maximum in gravel, where it may extend to 2 m in depth.

Prolonged freezing gives rise to shattering in the frozen layer, fracturing taking place along joints and cracks. Frost shattering, due to ice action in Pleistocene times, has been found to extend to depths of 30 m in the Chalk and to 12 m in the Borrowdale Volcanic Series in England, respectively. In this way, the concerned rock suffers a reduction in bulk density and an increase in deformability and permeability. Fretting and spalling are particularly rapid where the rock is closely fractured. Frost shattering may be concentrated along certain preferred planes if joint patterns are suitably oriented. Preferential opening takes place most frequently in those joints that run more or less parallel with the ground surface. Silt and clay frequently occupy the cracks in frost-shattered ground, down to appreciable depths, having been deposited by melt water. Their presence may cause stability problems.

Stress relief following the disappearance of ice on melting may cause enlargement of joints. This may aid failure on those slopes that were over-steepened by glaciation.

Stone polygons are common frozen-ground phenomena, and fossil forms are found in Pleistocene strata. They consist of marginal rings of stone that embrace mounds of finer material. Their diameters range up to 12 m.

Frost wedging is one of the chief factors of mechanical weathering in tundra regimes. Frozen soils often display a polygonal pattern of cracks. Individual cracks may be 1.2 m wide at their top, may penetrate to depths of 10 m and may be some 12 m apart. They form when, because of exceptionally low temperatures, shrinkage of the ground occurs. Ice wedges occupy these cracks and cause them to expand. When the ice disappears, an ice wedge pseudomorph is formed by sediment, frequently sand, filling the crack.

Ground may undergo notable disturbance as a result of mutual interference of growing bodies of ice or from excess pore water pressures developed in confined water-bearing lenses. Involutions are plugs, pockets or tongues of highly disturbed material, generally possessing inferior geotechnical properties, which have intruded into the overlying layers. They are formed as a result of hydrostatic uplift in water trapped under a refreezing surface layer. They are usually confined to the active layer. Involutions and ice wedge pseudomorphs usually mean that one material suddenly replaces another. This can cause problems in shallow excavations.

The movement downslope as a viscous flow of saturated rock waste is referred to as solifluction. It probably is the most significant process of mass wastage in tundra regions. Such movement can take place down slopes with gradients as low as 2°. The movement is extremely slow, most measurements showing rates ranging between 10 and 300 mm per year. Solifluction deposits commonly consist of gravels, which are characteristically poorly sorted, sometimes gap graded and poorly bedded. These gravels consist of fresh, poorly worn, locally derived material. Individual deposits are rarely more than 3 m thick and frequently display flow structures. Sheets and lobes of solifluction debris, transported by mudflow activity, are commonly found at the foot of slopes. These materials may be reactivated by changes in drainage, by stream erosion, by sediment overloading or during construction operations. Solifluction sheets may be underlain by slip surfaces, the residual strength of which controls their stability.

Periglacial action accelerates hill creep, the latter being particularly well developed on thinly bedded or cleaved rocks. Creep material may give way to solifluction deposits on approaching the surface. The creep deposits consist mainly of flat rock fragments oriented parallel with the hillside and are interrupted by numerous shallow slips.

Oversteepening of glaciated valleys and melt water channels occur when ground is stabilized by deep permafrost or supported by ice masses. Frost sapping at the bottom of scarp features also causes oversteepening. When the support disappears, the oversteepened slopes become potentially unstable. Melt water in a shattered rock mass gives rise to an increase in pore water pressures that, in turn, leads to movement or instability along bedding planes and joints. Increase in the moisture content of cohesive material brings about a reduction in its strength and may cause it to swell, thereby aggravating the instability, due to oversteepening, in the near-surface zone. As a result, landsliding on a large scale is associated with such oversteepened slopes.

The solubility of carbon dioxide in water varies inversely with temperature, for example, it is 1.7 times greater at 0°C than at 15°C. Accordingly, cold melt waters have frequently had a strong leaching effect on calcareous rocks. Some pipes and sinkholes in chalk may have been produced by such melt waters. The problem of buried pipes and sinkholes in chalk is aggravated by the frequent absence of surface evidence. They are often undetected by conventional site investigation.

Wind Action and Desert Landscapes

By itself, wind can only remove uncemented rock debris or soil, which it can perform more effectively if the material is dry rather than wet. But once armed with particles, the wind becomes a noteworthy agent of abrasion. The size of the particles that the wind can transport depends on the strength of the wind as well as the shape and weight of the particles. The distance that the wind, given that its velocity remains constant, can carry particles depends principally on their size.

Wind Action

Wind erosion takes place when air pressure overcomes the force of gravity on surface particles. At first, particles are moved by saltation. The impact of saltating particles on others may cause them to move by creep, saltation or suspension. Saltation accounts for three-quarters of the grains transported by wind, most of the remainder being carried in suspension, the rest are moved by creep or traction. Saltating grains may rise to a height of up to 2 m, their trajectory then being flattened by faster-moving air and tailing off as the grains fall to the ground. The length of the trajectory is roughly ten times the height.

One of the most important factors in wind erosion is its velocity. Its turbulence, frequency, duration and direction are also important. As far as the mobility of particles is concerned, the important factors are their size, shape and density. It would appear that particles less than 0.1 mm in diameter are usually transported in suspension, those between 0.1 and 0.5 mm are normally transported by saltation and those larger than 0.5 mm tend to be moved by traction or creep. Grains with a specific gravity of 2.65, such as quartz sand, are most susceptible to wind erosion in the size range 0.1 to 0.15 mm. A wind blowing at 12 km h^{-1} will move grains of 0.2 mm diameter – a lesser velocity will keep the grains moving.

Because wind can only remove loose particles of a limited size range, if erosion is to proceed beyond the removal of such particles, then the remaining material must be sufficiently broken down by other agents of erosion or weathering. Material that is not sufficiently reduced in size seriously inhibits further wind erosion. Obviously, removal of fine material leads to a proportionate increase in that of larger size that cannot be removed. The latter affords increasing protection against continuing erosion, and a wind-stable surface is eventually created. Binding agents, such as silt, clay and organic matter, hold particles together, making wind erosion more difficult. Soil moisture also contributes to cohesion between particles.

Generally, a rough surface tends to reduce the velocity of the wind immediately above it. Consequently, particles of a certain size are not as likely to be blown away as they would on a smooth surface. Even so, Bagnold (1941) found that grains of sand less than 0.03 mm in diameter were not lifted by the wind if the surface on which they lay was smooth. On the other hand, particles of this size can easily remain suspended by the wind. The longer the surface distance over which a wind can blow without being interrupted, the more likely it is to attain optimum efficiency.

There are three types of wind erosion, namely, deflation, attrition and abrasion. Deflation results in the lowering of land surfaces by loose unconsolidated rock waste being blown away by the wind. The effects of deflation are seen most acutely in arid and semi-arid regions. For example, basin-like depressions are formed by deflation in the Sahara and Kalahari deserts.

However, downward lowering is almost invariably arrested when the water table is reached since the wind cannot readily remove moist rock particles. What is more, deflation of sedimentary material, particularly alluvium, creates a protective covering if the material contains pebbles. The fine particles are removed by the wind, leaving a surface formed of pebbles that are too large to be blown away. The suspended load carried by the wind is comminuted further by attrition, turbulence causing the particles to collide vigorously with one another.

When the wind is armed with grains of sand, it possesses great erosive force, the effects of which are best displayed in rock deserts. Accordingly, any surface subjected to prolonged attack by wind-blown sand is polished, etched or fluted. Abrasion has a selective action, picking out the weaknesses in rocks. For example, discontinuities are opened and rock pinnacles developed. Since the heaviest rock particles are transported near to the ground, abrasion is there at its maximum and rock pedestals may be formed. In deserts, flat smoothed surfaces produced by wind erosion are termed desert pavements.

The differential effects of wind erosion are illustrated in areas where alternating beds of hard and soft rock are exposed. If strata are tilted steeply, a ridge and furrow relief develops, because soft rocks are more readily worn away than hard. Such ridges are called yardangs. Conversely, when an alternating series of hard and soft rocks are more or less horizontally bedded, features known as zeugens are formed. In such cases, the beds of hard rock act as resistant caps, affording protection to the soft rocks beneath. Nevertheless, any weaknesses in the hard caps are picked out by weathering, and the caps are breached eventually, exposing the underlying soft rocks. Wind erosion rapidly eats into the latter and, in the process, the hard cap is undermined. As the action continues, tabular masses, known as mesas and buttes, are left in isolation (Fig. 3.32).

Desert Dunes

About one-fifth of the land surface of the Earth is desert. Approximately four-fifths of this desert area consists of exposed bedrock or weathered rock waste. The rest is mainly covered with deposits of sand. Bagnold (1941) recognized five main types of sand accumulations, namely, sand drifts and sand shadows, whalebacks, low-scale undulations, sand sheets and true dunes. He further distinguished two kinds of true dunes, the barkhan and the seif (Fig. 3.33a and b).

Several factors control the form that an accumulation of sand adopts. Firstly, there is the rate at which sand is supplied; secondly, there is wind speed, frequency and constancy of direction; thirdly, there is the size and shape of the sand grains; and fourthly, there is the nature of the surface across which the sand is moved. Sand drifts accumulate at the exits of the gaps in the landscape through which wind is channelled and are extended downwind.

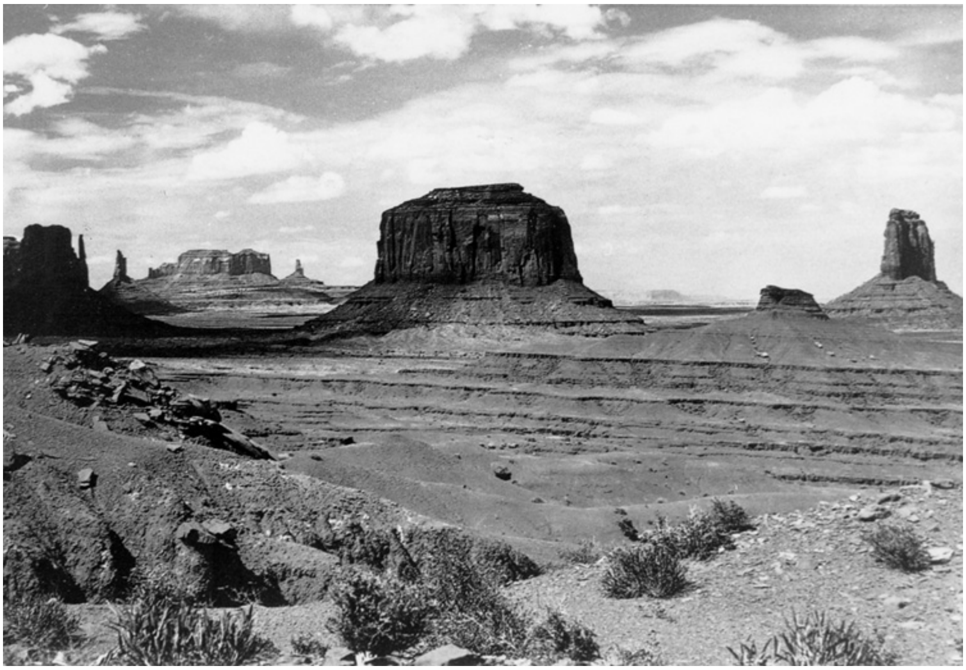


Figure 3.32

Buttes and mesas, Monument Valley, Utah.

However, such drifts, unlike true dunes, are dispersed if they are moved downwind. Whalebacks are large mounds of comparatively coarse sand that are thought to represent the relics of seif dunes. Presumably, the coarse sand is derived from the lower parts of seifs, where accumulations of coarse sand are known to exist. These features develop in regions devoid of vegetation. By contrast, undulating mounds are found in the peripheral areas of deserts where the patchy cover of vegetation slows the wind and creates sand traps. Large undulating mounds are composed of fine sand. Sand sheets are also developed in the marginal areas of deserts. These sheets consist of fine sand that is well sorted, indeed they often present a smooth surface that is capable of resisting wind erosion. A barkhan is crescentic in outline and is orientated at right angles to the prevailing wind direction, whereas a seif is a long, ridge-shaped dune running parallel to the direction of the wind. Seif dunes are much larger than barkhans, they may extend lengthwise for up to 90 km and reach heights up to 100 m. Barkhans are rarely more than 30 m in height, and their width is usually about 12 times their height. Generally, seifs occur in great numbers, running approximately equidistant from each other, with the individual crests separated from each other by distances of 30 to 500 m.

It is commonly believed that sand dunes come into being where some obstacle prevents the free flow of sand, sand piling up on the windward side of the obstacle to form a dune. But in

(a)

(b)

Figure 3.33

(a) Barkhan dunes, Death Valley, California. (b) Seif dunes, near Sossusvlei, Namibia.

areas in which there is exceptionally low rainfall and, therefore, little vegetation to impede the movement of sand, observation has revealed that dunes develop most readily on flat surfaces that are devoid of large obstacles. It would seem that where the size of the sand grains varies or where a rocky surface is covered with pebbles, dunes grow over areas of width greater than 5 m. Such patches exert a frictional drag on the wind, causing eddies to blow sand towards them. Sand is trapped between the larger grains or pebbles, and an accumulation results. If a surface is strewn with patches of sand and pebbles, deposition takes place over the pebbles. However, patches of sand exert a greater frictional drag on strong winds than do patches of pebbles, and deposition under such conditions thus takes place over the former. When strong winds sweep over a rough surface, they become transversely unstable, and barkhans may develop.

Longitudinal dunes may develop from barkhans. Suppose that for some reason the tails of a barkhan become fixed, for example, by vegetation or by the water table rising to the surface. Then the wind continues to move the central part until the barkhan eventually loses its convex shape, becoming concave towards the prevailing wind. As the central area becomes further extended, the barkhan may split. The two separated halves are rotated gradually by the eddying action of the wind until they run parallel to one another, in line with the prevailing wind direction. Dunes that develop in this manner are often referred to as blow-outs.

Seif dunes appear to form where winds blow in two directions, that is, where the prevailing winds are gentle and carry sand into an area, the sand then being driven by occasional strong winds into seif-like forms. Seifs may also develop along the bisectrix between two diagonally opposed winds of roughly equal strength. Because of their size, seif dunes can trap coarse sand much more easily than can barkhans. This material collects along the lower flanks of the dune. Indeed, barkhans sometimes occur in the troughs of seif dunes. On the other hand, the trough may be floored by bare rock.

Salt Weathering and Duricrusts

Salt weathering is characteristic of hot deserts and leads to rock disintegration. This is brought about as a result of the stresses set up in the pores, joints and fissures in rock masses due to the growth of salts, the hydration of particular salts and the volumetric expansion that occurs due to the high diurnal range of temperature. The aggressiveness of the ground depends on the position of the water table and the capillary fringe above in relation to the ground surface; the chemical composition of the groundwater and the concentration of salts within it; the type of soil and the soil temperature. The pressures produced by the crystallization of salts in small pores are appreciable. Some common salts hydrate and dehydrate relatively easily in response to changes in temperature and humidity. Hydration increases the volume of the salts and, hence, develops pressure within the pores or cracks in rocks. Such increase in volume may be appreciable. What is more, some of these changes may take place rapidly. The hydration, dehydration and rehydration of hydrous salts may occur several times throughout a year, and depend on the temperature and relative humidity conditions on the one hand and dissociation vapour pressures of the salts on the other. New layers of minerals can be formed within months, and thin layers can be dissolved just as quickly. In fact, Obika et al. (1989) indicated that crystallization and hydration–dehydration thresholds of the more soluble salts such as sodium chloride, sodium carbonate, sodium sulphate and magnesium sulphate may be crossed at least once daily. Because of the high rate of evaporation in hot arid regions, the capillary rise of near-surface groundwater is normally very pronounced (Al Sanad et al., 1990). Salts are precipitated on the ground surface in the form of effluorescences and also are precipitated in the upper layers of the soil. Nonetheless, the occurrence of salts is extremely variable from place to place.

Salt weathering also attacks structures and buildings, leading to cracking, spalling and disintegration of concrete, brick and stone. One of the most notable forms of damage to buildings and structures is that attributable to sulphate attack (Robinson, 1995). The most serious damage caused to brickwork and limestone and sandstone building stone occurs in low-lying salinas or sabkhas (see Chapter 5), where saline groundwater occurs at shallow depth, giving rise to aggressive ground conditions. Salt weathering of bituminous paved roads built over areas where saline groundwater is at or near the surface is likely to result in notable signs of damage such as heaving, cracking, blistering, stripping, potholing, doming and disintegration (Blight, 1994).

Duricrust or pedocrete is a surface or near-surface hardened accumulation or encrusting layer, formed by precipitation of salts on evaporation of saline groundwater. Duricrusts may be composed of calcium or magnesium carbonate, gypsum, silica, alumina or iron oxide, or even halite, in varying proportions. It may occur in a variety of forms, ranging from a few millimetres in thickness to over a metre. A leached cavernous, porous or friable zone is frequently found beneath the duricrust. Pedocretes refer to hardened surfaces that usually occur on hard rock (e.g. calcrete), whereas duricrusts are softer accumulations (e.g. gypcrust) that are usually found in salt playas, salinas or sabkhas. Locally, especially near the coast, sands may be cemented with calcrete to form cap-rock or miliolite. Desert fill often consists of mixtures of nodular calcrete, calcrete fragments and drifted sand.

Stream Action in Arid and Semi-Arid Regions

It must not be imagined that stream activity plays an insignificant role in the evolution of landscape in arid and semi-arid regions. Admittedly, the amount of rainfall occurring in arid regions is small and irregular, whereas that of semi-arid regions is markedly seasonal. Nevertheless, it commonly falls in both instances as intense and often violent showers. The result is that the river channels frequently cannot cope with the amount of rain water, and extensive flooding takes place. These floods develop with remarkable suddenness and either form raging torrents, which tear their way down slopes excavating gullies as they go, or they may assume the form of sheet floods. Dry wadis are rapidly filled with swirling water and, thereby, are enlarged. However, such floods are short lived since the water soon becomes choked with sediment and the consistency of the resultant mudflow eventually reaches a point when further movement is checked. Much water is lost by percolation, and mudflows are also checked where there is an appreciable slackening in gradient.

Some of the most notable features produced by stream action in arid and semi-arid regions are found in intermontane basins, that is, where mountains circumscribe a basin of inland drainage. The rain that falls on the encircling mountains causes flooding and active erosion. Mechanical weathering plays a significant role in the mountain zone. Boulders, 2 m or more in diameter, are found in gullies that cut the mountain slopes, whereas finer gravels, sands and muds are washed downstream.

Alluvial cones or fans, which consist of irregularly sorted sediment, are found along the foot of the mountain belt where it borders the pediment, the marked change in gradient accounting for rapid deposition. The particles composing the cones are almost all angular in shape, boulders and cobbles being more frequent upslope, grading downslope into fine gravels. The cones have a fairly high permeability. When these alluvial cones merge into one another, they form a bahada. The streams that descend from the mountains rarely reach the centre of the basin since they become choked by their own deposits and split into numerous distributaries that spread a thin veneer of gravels over the pediment.

Although stream flow on alluvial cones is ephemeral, flooding nevertheless can constitute a serious problem, occurring along the margins of the main channels and in the zone deposition beyond the ends of supply channels. The flood waters are problematic because of their high velocities and their variable sediment content. They also have a tendency to change locations with successive floods, abandoning and creating channels in a relatively short time.

Hydrocompaction may occur on alluvial cones. The dried surface layer of these cones may contain many voids. Percolating water frequently reduces the strength of the material that, in turn, reduces the void space. This gives rise to settlement or hydrocompaction.

Pediments in semi-arid regions are graded plains cut by the lateral erosion of ephermeral streams. They are adjusted to dispose off water efficiently, and the heavy rainfall characteristic of semi-arid regions means that this is often in the form of sheet wash. Although laminar flow occurs during sheet wash, it yields to turbulent flow as the flowing water deepens.

The latter possesses much greater erosive power and occurs during and immediately after heavy rainfall. This is why these pediments carry only a thin veneer of rock debris. With a lesser amount of rainfall, there is insufficient water to form sheets, and it is confined to rills and gullies. Aeolian and fluvial deposits, notably sand, also may be laid down in the intermediate zone between the pediment and the central depression or playa. However, if deflation is active, this zone may be barren of sediments. Sands are commonly swept into dunes and the resultant deposits are cross-bedded.

The central area of a basin is referred to as the playa, and it sometimes contains a lake (Fig. 3.34). This area is covered with deposits of sand, silt, clay and evaporites. The silts and clays often contain crystals of salt whose development comminutes their host. Silts usually exhibit ripple marks, whereas clays are frequently laminated. Desiccation structures such as mudcracks are developed on an extensive scale in these fine-grained sediments. If there is a playa lake and it has contracted to leave a highly saline tract, then this area is termed a salina. The capillary rise generally extends to the surface, leading to the formation of a salt crust. Where the capillary rise is near to, but normally does not reach the surface, desiccation ground patterns provide an indication of its closeness.

Figure 3.34

View across the playa of Death Valley, California. Note the alluvial fans merging into bahadas at the foot of the far mountain range.

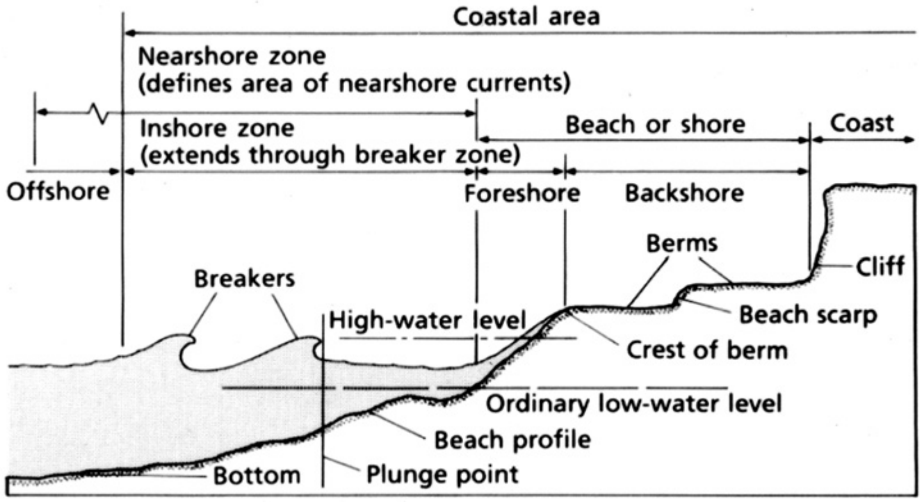


Figure 3.35

Terminology of beach features.

Coasts and Shorelines

The shore zone can be divided into the coast, the shore and the offshore (Fig. 3.35). The coast has been defined as the land immediately behind the cliffs, whereas the shore is regarded as that area between the base of the cliffs and low-water mark. That area that extends seawards from the low-water mark is termed the offshore. The shore itself is further divided into foreshore and backshore, the former embracing the intertidal zone, while the latter extends from the foreshore to the cliffs.

Wave Action

When wind blows across the surface of deep water, it causes an orbital motion of the water particles in the plane, normal to the wind direction. Because adjacent particles are at different stages of their circular course, a wave is produced. The motion is transmitted to the water beneath the surface, but the orbitals are rapidly reduced in size with increasing depth, and the motion dies out at a depth equal to that of the wavelength (Fig. 3.36). There is no progressive forward motion of the water particles in such a wave, although the form of the wave profile moves rapidly in the direction in which the wind is blowing. Such waves are described as oscillatory waves.

The parameters of a wave are the wavelength, L, that is, the horizontal distance between each crest or each trough, the wave height, H, the vertical distance between the crest and

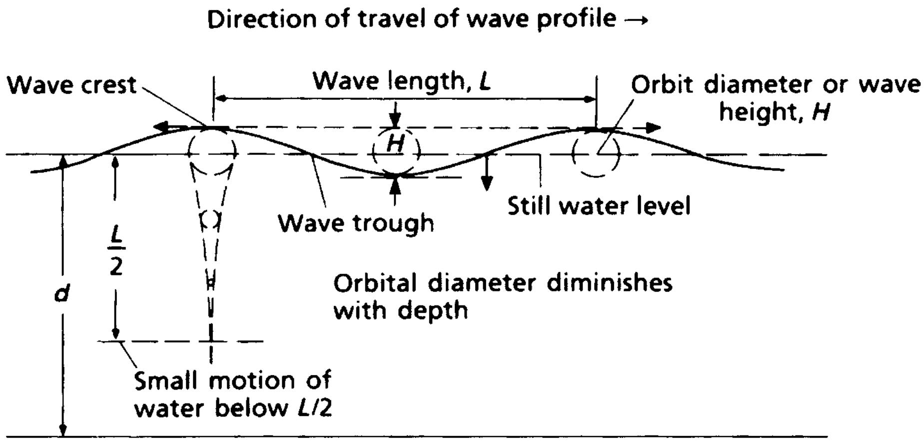


Figure 3.36

Orbital motion of water particles during the passage of an idealized sinusoidal wave in deep water. The orbital diameter decreases with depth and disappears at a depth of approximately one half the wave length.

the trough, and the wave period, T, the time interval between the passage of successive wave crests. The rate of propagation of the wave form is the wavelength divided by the wave period. The height and period of waves are functions of the wind velocity, the fetch (i.e. the distance over which the wind blows) and the length of time for which the wind blows.

Fetch is the most important factor determining wave size and efficiency of transport. Winds of moderate force that blow over a wide stretch of water generate larger waves than do strong winds that blow across a short reach. Where the fetch is less than 32 km, the wave height increases directly with, and the wave period increases as the square root of, the wind velocity. Long waves only develop where the fetch is large, for instance, the largest waves are generated in the southern oceans where their lengths may exceed 600 m and their periods may be greater than 20 s. Usually, wavelengths in the open sea are less than 100 m, and the speed of propagation is approximately 50 km h^{-1}.

Those waves that are developed in storm centres in the centre of an ocean may journey to its limits. This explains why large waves may occur along a coast during fine weather.

Waves frequently approach a coastline from different areas of generation. If they are in opposition, then their height is decreased, whereas their height is increased if they are in phase.

Four types of waves have been distinguished, that is, forced, swell, surf and forerunners. Forced waves are those formed by the wind in the generating area; they are usually irregular. On moving out of the area of generation, the waves become long and regular. They are then referred to as swell or free waves. As these waves approach a shoreline, they feel the bottom, which disrupts their pattern of motion, changing them from oscillation to translation waves; in other words, they break into surf. The longest and lowest waves are termed forerunners or swell.

The breaking of a wave is influenced by its steepness, the slope of the sea floor and the presence of an opposing or supplementary wind. When waves enter waters equal in depth to their wavelength, they begin to feel the bottom and their length decreases, while their height increases. Their velocity of travel or celerity, c, is also reduced, in accordance with the expression:

$$c = \left[\frac{gL}{2\pi} \tanh\left(\frac{2\pi Z}{L} \right) \right]^{1/2} \tag{3.9}$$

where L is the wavelength, Z is the depth of the water and g is acceleration due to gravity. As a result, the wave steepens until the wave train consists of peaked crests separated by relatively flat troughs. The wave period, however, remains constant. Wave steepening accelerates towards the breaker zone, and the wave height grows to several times what it was in deep water. Three types of breaking waves are recognized, plunging, spilling and surging or swash. Plunging breakers collapse when their wave height is approximately equal to the depth of the water. They topple suddenly and fall with a crash. They are usually a consequence of long, low swell, and their formation is favoured by opposing winds. Spilling breakers begin to break when the wave height is just over one-half of the water depth, and they do so gradually over some distance. Generally, they result from steep wind waves, and they commonly occur when the wind is blowing in the direction of wave propagation. Surging breakers or swash rush up the beach and are usually encountered on beaches with steep profiles. The term backwash is used to describe the water that subsequently descends the beach slope.

Four dynamic zones have been recognized within the nearshore current system of the beach environment. They are the breaker zone, the surf zone, the transition zone and the swash zone. The breaker zone is that in which waves break. The surf zone refers to that region between the breaker zone and the effective seaward limit of the backwash. The presence and width of a surf zone is primarily a function of the beach slope and tidal phase. Those beaches that have gentle foreshore slopes are often characterized by wide surf zones during all tidal phases, whereas steep beaches seldom possess this zone. The transition zone includes that region where backwash interferes with the water at the leading edge of the surf zone, and it is characterized by high turbulence. That region where water moves up and down the beach is termed the swash zone.

Swash tends to pile water against the shore. After flowing parallel to the beach, the water runs back to the sea in narrow flows called rip currents. In the neighbourhood of the breaker zone, rip currents extend from the surface to the floor, whereas in the deeper reaches, they override the bottom water that still maintains an overall onshore motion. The positions of rip currents are governed by submarine topography, coastal configuration and the height and period of the waves. They frequently occur on the up-current sides of points and on either side of convergences where the water moves away from the centre of convergence and turns seawards. The onshore movement of water by wave action in the breaker zone; the lateral transport by longshore currents in the breaker zone; the seaward return of flow as rip currents through the surface zone; and the longshore movement in the expanding head of a rip current, all form part of the nearshore circulation system.

Tides may play an important part in beach processes. In particular, the tidal range is responsible for the area of the foreshore over which waves are active. Tidal streams are especially important where a residual movement resulting from differences between ebb and flood occurs, and where there is abundant loose sediment for the tidal streams to transport. They are frequently fast enough to carry coarse sediment, but the features, notably bars, normally associated with tidal streams in the offshore zone or in tidal estuaries usually consist of sand. Therefore, these features only occur where sufficient sand is available. In quieter areas, tidal mud flats and salt marshes are developed, where the tide ebbs and floods over large flat expanses, depositing muddy material. Mud also accumulates in runnels landward of high ridges, in lagoons and on the lower foreshore where shelter is provided by offshore bars.

Marine Erosion

Coasts undergoing erosion display two basic elements of the coastal profile, namely, the cliff and the bench or platform. In any theoretical consideration of the evolution of a coastal profile, it is assumed that the coast is newly uplifted above sea level. After some time, a wave-cut notch may be excavated, and its formation intensifies marine erosion in this narrow zone. The development of a notch varies according to the nature of the rock in which excavation is proceeding, for example, it may be present if the sediments are unconsolidated or if the bedding planes dip seawards. Where a notch develops, it gives rise to a bench, and the material above is undermined and collapses to form a cliff face.

Pot holes are common features on most wave-cut benches; they are excavated by pebbles and boulders being swilled around in depressions. As they increase in diameter, they coalesce and so lower the surface of the bench. The debris produced by cliff recession gives rise to a rudimentary beach. In tidal seas, the base of a cliff is generally at high-tide level, whereas in non-tidal seas, it usually is above still-water level.

As erosion continues, the cliff increases in height, and the bench widens. The slope adopted by the bench below sea level is determined by the ratio of the rate of erosion of the slope to the recession of the cliff. A submarine accumulation terrace forms in front of the bench and is extended out to sea. Because of the decline in wave energy consequent on the formation of a wide flat bench, the submarine terrace deposits may be spread over the lower part of the bench in the final stages of its development. The rate of cliff recession is therefore retarded, and the cliff becomes gently sloping and moribund.

If the relationship between the land and sea remains constant, then erosion and, consequently, the recession of land beneath the sea is limited. Although sand can be transported at a depth of half the length of storm waves, bedrock is abraded at half this depth or less. As soon as a submarine bench slope of 0.01 to 0.05 is attained, bottom abrasion generally ceases, and any further deepening is brought about by organisms or chemical solution. Such rock destruction can occur at any depth, but the floor can only be lowered where currents remove the altered material.

The nature of the impact of a wave on a coastline depends to some extent on the depth of the water and partly on the size of the wave. The vigour of marine action drops sharply with increasing depth from the water surface, in fact, at approximately the same rate as the decline in the intensity of wave motion. Erosion is unlikely to take place at a depth of more than 60 m along the coast of an open sea and at less than that in closed seas.

If deep water occurs alongside cliffs, then waves may be reflected without breaking and, in so doing, they may interfere with incoming waves. In this way, clapotis (standing waves that do not migrate) are formed. It is claimed that the oscillation of standing waves causes an alternate increase and decrease of pressure along discontinuities in rocks that occur in that part of the cliff face below the waterline. Also, when waves break, a jet of water is thrown against the cliff at approximately twice the velocity of the wave and, for a few seconds, this increases the pressure within the discontinuities. Such action gradually dislodges blocks of rock.

Those waves with a period of approximately 4 s are usually destructive, whereas those with a lower frequency, that is, a period of about 7 s or over, are constructive. When high-frequency waves collapse, they form plunging breakers, and the mass of water is accordingly directed downwards at the beach. In such instances, swash action is weak and, because of the high frequency of the waves, is impeded by the backwash. As a consequence, material is removed from the top of the beach. The motion within waves that have a lower frequency is more elliptical and produces a strong swash that drives material up the beach. In this case, the backwash is reduced in strength because water percolates into the beach deposits and, therefore, little material is returned down the beach. Although large waves may throw material above the high water level and thus act as constructive agents, they nevertheless have an overall tendency to erode the beach, whereas small waves are constructive.

Swash is relatively ineffective compared to backwash on some shingle beaches. This frequently leads to very rapid removal of the shingle from the foreshore into the deeper water beyond the breakpoint. Storm waves on such beaches, however, may throw some pebbles to considerable elevations above mean sea level, creating a storm-beach ridge and, because of the rapid percolation of water through the shingle, backwash does not remove these pebbles. On the other hand, when steep storm waves attack a sand beach, they are usually destructive and, the coarser the sand, the greater the quantity that is removed. Some of this sand may form a submarine bar at the break point, whereas some is carried into deeper water offshore. It is by no means a rarity for the upper part of a beach to be removed by storm waves.

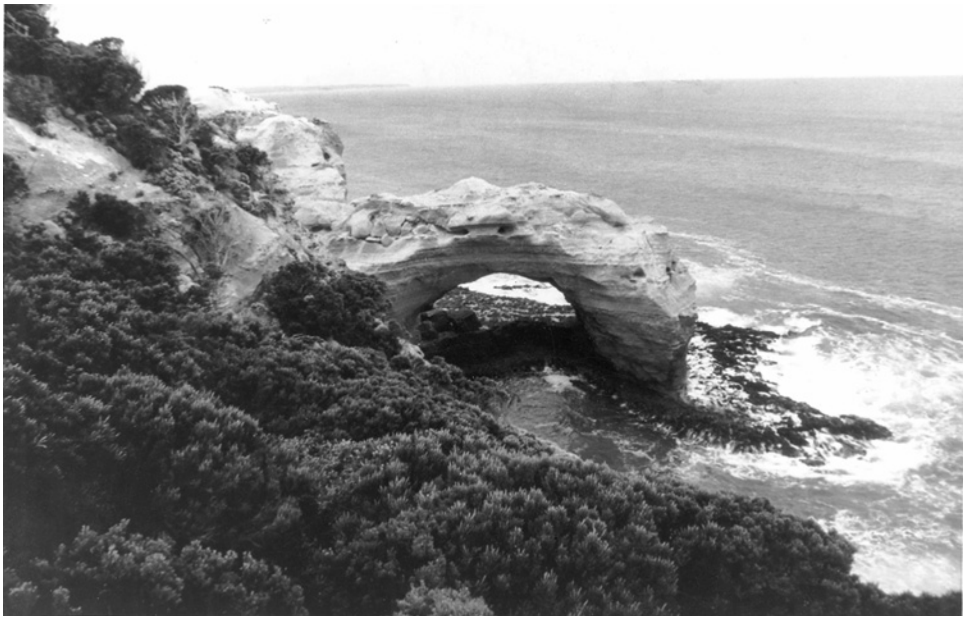
Waves usually leave little trace on massive smooth rocks except to polish them. However, where there are irregularities or projections on a cliff face, the upward spray of breaking waves quickly removes them (the force of upward spray along a seawall can be as much as 12 times that of the horizontal impact of the wave).

The degree to which rocks are traversed by discontinuities affects the rate at which they are removed by marine erosion. In particular, the attitude of joints and bedding planes is important. Where the bedding planes are vertical or dip inland, the cliff recedes vertically under marine attack. But if beds dip seawards, blocks of rock are dislodged more readily, since the removal of material from the base of the cliff means that the rocks above lack support and tend to slide into the sea. Joints may be enlarged into deep narrow inlets. Marine erosion also is concentrated along fault planes.

The height of a cliff is another factor that influences the rate at which coastal erosion takes place. The higher the cliff, the more material falls when its base is undermined. This, in turn, means that a greater amount of debris has to be broken down and removed before the cliff is once more attacked with the same vigour.

Erosive forms of local relief include such features as wave-cut notches, caves, blowholes, marine arches and stacks (Fig. 3.37). Marine erosion is concentrated in areas along a coast where the rocks offer less resistance. Caves and small bays or coves are excavated where the rocks are softer or strongly jointed. At the landward end of large caves, there is often an opening to the surface, through which spray issues, which is known as a blowhole. Blowholes are formed by the collapse of jointed blocks loosened by wave-compressed air. A marine arch is developed when two caves on opposite sides of a headland unite. When the arch falls, the isolated remnant of the headland is referred to as a stack.

Wave refraction is the process whereby the direction of wave travel changes because of changes in the topography of the nearshore sea floor. When waves approach a straight beach at an angle, they tend to swing parallel to the shore due to the retarding effect of the shallowing water. At the break point, such waves seldom approach the coast at an angle



(a)

(b)

Figure 3.37 *(Continued)*

(a) Marine arch, south coast of California. (b) Stacks, the Seven Apostles, off the southern coast of Victoria, Australia.

(c)
Figure 3.37, cont'd

(c) Blowhole, Pancake Rocks, South Island, New Zealand.

exceeding 20°, irrespective of the offshore angle to the beach. As the waves break, they develop a longshore current, indeed wave refraction is often the major factor in dictating the magnitude and direction of longshore drift. Wave refraction also is responsible for the concentration of erosion on headlands that leads to a coast being gradually smoothed in outline. As waves approach an irregular shoreline refraction causes them to turn into shallower water so that the wave crests run roughly parallel to the depth contours. Along an indented coast, shallower water is met with first off headlands. This results in wave convergence and an increase in wavelength, with wave crests becoming concave towards headlands. Conversely, where waves move towards a depression in the sea floor, they diverge, are decreased in height and become convex towards the shoreline.

Constructive Action of the Sea

Beaches may be supplied with sand that is derived almost entirely from the adjacent sea floor, although in some areas, a larger proportion is produced by cliff erosion. During periods of low waves, the differential velocity between onshore and offshore motion is sufficient to move sand onshore except where rip currents are operational. Onshore movement is particularly notable when long-period waves approach a coast, whereas sand is removed from the foreshore during high waves of short period.

Table 3.2. Average beach slopes compared with sediment diameters

Beach sediment	Particle size (mm)	Average slope of beach
Fine sand	0.06–0.20	1–3°
Medium sand	0.2–0.6	3–5°
Coarse sand	0.6–2.0	5–9°
Fine gravel	2.0–6.0	9–12°
Medium gravel	6.0–20.0	12–17°
Coarse gravel	20.0–60.0	17–24°
Cobbles	60.0–200.0	Over 24°

The beach slope is produced by the interaction of swash and backwash. It is also related to the grain size and permeability of the beach (Table 3.2). For example, the loss of swash due to percolation into beaches composed of grains that are 4 mm in median diameter is ten times greater than into those where the grains average 1 mm. As a result, there is almost as much water in the backwash on fine sandy beaches as there is in the swash, so the beach profile is gentle and the sand is hard packed.

The waves that produce the most conspicuous constructional features on a shingle beach are storm waves. A small foreshore ridge develops on a shingle beach at a limit of the swash when constructional waves are operative. Similar ridges or berms may form on a beach composed of coarse sand. Berms represent a marked change in slope and usually occur a small distance above the high water mark. However, they may be overtopped by high spring tides. Berms are not such conspicuous features on beaches of fine sand. Greater accumulation occurs on coarse sandy beaches because their steeper gradient means that the wave energy is dissipated over a relatively narrow width of beach.

Beach cusps and sandbars are constructional features of small size. The former are commonly found on shingle beaches. They consist of a series of ridges composed of shingle that are separated by troughs in which finer material occurs. Sandbars are characteristic of tideless seas. Their location is related to the breakpoint that, in turn, is related to wave size. Consequently, more than one bar may form, the outermost being attributable to storm waves, the inner to normal waves. On tidal beaches, the breakpoint migrates over a wide zone; hence, sandbars do not form and they may be replaced by ripple marks.

When waves move parallel to the coast, they simply move sand and shingle up and down the beach. On the other hand, when they approach the coast at an angle, material is moved up the beach by the swash in the direction normal to that of wave approach, and it is then rolled down the steepest slope of the beach by the backwash. Consequently, material is moved in a zigzag path along the beach. This is known as longshore or littoral drift. Such action can transport pebbles appreciable distances along a coast. The duration of movement along a coastline is dependent on the direction of the dominant winds. An indication of the direction of longshore drift is provided by the orientation of spits along a coast.

Material may be supplied to the littoral sediment budget by coastal erosion, by feed from offshore or by contributions from rivers. After sediment has been distributed along the coast by longshore drift, it may be deposited in a sediment reservoir and, therefore, lost from the active environment. Sediment reservoirs formed offshore take the form of bars where the material is in a state of dynamic equilibrium, but from which it may easily re-enter the system. Dunes are the commonest types of onshore reservoirs, from which sediment is less likely to re-enter the system.

Bay-head beaches are one of the commonest types of coastal deposits, and they tend to straighten a coastline. Wave refraction causes longshore drift to move from headlands to bays where sediments are deposited. Marine deposition also helps straighten coastlines by building beach plains.

Spits are deposits that grow out from the coast. They are supplied with material chiefly by longshore drift. Their growth is spasmodic and alternates with episodes of retreat. The stages in the development of many complex spits and forelands are marked by beach ridges that frequently are continuous over long distances. While longshore drift provides the material for construction, their development results from spasmodic progradation by frontal wave accretion during major storms. The distal end of a spit is frequently curved (Fig. 3.38). Those spits that extend from the mainland to link up with an island are known as tombolas.

Bay-bars are constructed across the entrance to bays by the growth of a spit being continued from one headland to another. Bays may also be sealed off if spits, which grow from both headlands, merge. If two spits extending from an island meet, they form a looped bar.

A cuspate bar arises either where a change in a direction of spit growth takes place so that it eventually joins the mainland again, or where two spits coalesce. If progradation occurs, then cuspate bars give rise to cuspate forelands (Fig. 3.39).

Offshore bars or barriers consist of ridges of shingle or sand. They usually project above sea level, extend for several kilometres and are located a few kilometres offshore.

Storm Surges and Tsunamis

Except where caused by failure of protection works, marine inundation is almost always attributable to severe meteorological conditions, giving rise to abnormally high sea levels, referred to as storm surges. A storm surge can be regarded as the magnitude of sea level

Figure 3.38

Hurst Castle Spit with recurved laterals, Hampshire, England.

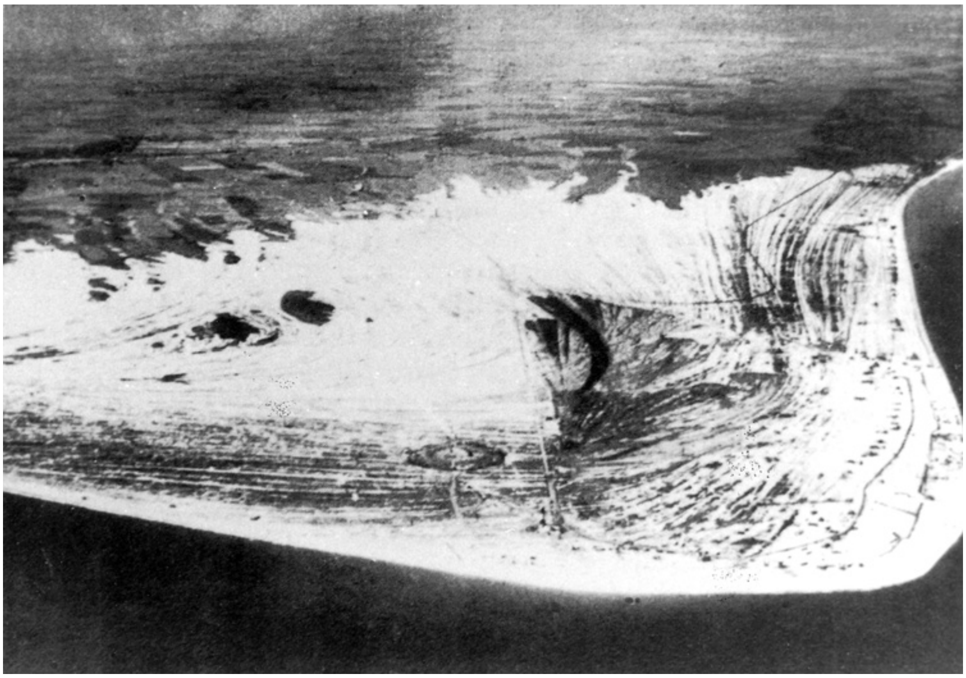


Figure 3.39

A cuspate foreland, Dungeness, Kent, England.

Figure 3.40

Flooded polder at St Philipsand, The Netherlands, January 1953.

along the shoreline that is above the normal seasonally adjusted high-tide level. Low pressure and driving winds during a storm may lead to marine inundation of low-lying coastal areas, particularly if they coincide with high spring tides. This is especially the case when the coast is unprotected. Floods may be frequent, as well as extensive where flood plains are wide and the coastal area is flat. Coastal areas that have been reclaimed from the sea and are below high-tide level are vulnerable if coastal defences are breached (Fig. 3.40). Storm surge risk is often associated with a particular season. The height and location of storm damage along a coast over a period of time, when analyzed, provides some idea of the maximum likely elevation of surge effects. The seriousness of the damage caused by storm surge tends to be related to the height and velocity of water movement.

Factors that influence storm surges include the intensity in the fall in atmospheric pressure, the length of water over which the wind blows, the storm motion and offshore topography. Obviously, the principal factor influencing storm surge is the intensity of the causative storm, the speed of the wind piling up the sea against the coastline. For instance, threshold windspeeds of approximately 120 km h^{-1} tend to be associated with central pressure drops of around 34 mbar. Normally, the level of the sea rises with reductions in atmospheric pressure associated with intense-low-pressure systems. In addition, the severity of a surge is influenced by the size and track of a storm and, especially in the case of open coastline surges, by the nearness of the storm track to the coastline. The wind direction and the length of fetch are also important, both determining the size and energy of waves. Because of the influence of the topography of the sea floor, wide shallow areas on the continental shelf are more susceptible to damaging surges than where the shelf slopes steeply. Surges are intensified by converging coastlines that exert a funnel effect as the sea moves into such inlets.

One of the most terrifying phenomena that occur along coastal regions is called tsunami, the inundation by a large mass of water (Fig. 3.41). Most tsunamis originate as a result of fault movement, generating earthquakes on the sea floor, although they also can be developed by submarine landslides or volcanic activity. However, even the effects of large earthquakes are relatively localized compared to the impact of tsunamis. As with other forms of waves, it is the energy of tsunamis that is transported, and not the mass. Oscillatory waves are developed with periods of 10 to 60 min that affect the whole column of water from the bottom of the ocean to the surface. Together with the magnitude of an earthquake and its depth of focus, the amount of vertical crustal displacement determine the size, orientation and

Figure 3.41

The northern end of Resurrection Bay at Seward, Alaska, after it had been affected by a tsunami. The epicentre of the earthquake was 75 km distant.

destructiveness of tsunamis. Due to the long period of tsunamis, the waves are of great length (e.g. 200 to 700 km in the open ocean, 50 to 150 km on the continental shelf). Therefore, it is almost impossible to detect tsunamis in the open ocean because their amplitudes (0.1 to 1.0 m) are extremely small in relation to their length. They can only be detected near the shore. Soloviev (1978) devised a classification of tsunami intensity, as given in Table 3.3.

Table 3.3. Scale of tsunami intensity (Soloviev 1978)

Intensity	Run-up height (m)	Description of tsunami	Frequency in Pacific Ocean
I	0.5	*Very slight*. Wave so weak as to be perceptible only on tidal gauge records.	One per hour
II	1	*Slight*. Waves noticed by people living along the shore and familiar with the sea. On very flat shores, waves are generally noticed.	One per month
III	1	*Rather large*. Generally noticed. Flooding of gently sloping coasts. Light sailing vessels carried away on shore. Slight damage to light structures situated near the coast. In estuaries, reversal of river flow for some distance upstream.	One per 8 months
IV	4	*Large*. Flooding of the shore to some depth. Light scouring on made ground. Embankments and dykes damaged. Light structures near the coast damaged. Solid structures on the coast lightly damaged. Large sailing vessels and small ships swept inland or carried out to sea. Coasts littered with floating debris.	One per year
V	8	*Very large*. General flooding of the shore to some depth. Quays and other heavy structures near the sea damaged. Light structures destroyed. Severe scouring of cultivated land and littering of the coast with floating objects, fish and other sea animals. With the exception of large ships, all vessels carried inland or out to sea. Large bores in estuaries. Harbour works damaged. People drowned, waves accompanied by a strong roar.	Once in 3 years
VI	16	*Disastrous*. Partial or complete destruction of man-made structures for some distance from the shore. Flooding of coasts to great depths. Large ships severely damaged. Trees uprooted or broken by the waves. Many casualties.	Once in 10 years

Free oscillations develop when tsunamis reach the continental shelf, which modify their form. Usually, the largest oscillation level is not the first but one of the subsequent oscillations. However, to the observer on the coast, a tsunami appears not as a sequence of waves but as a quick succession of floods and ebbs (i.e. a rise and fall of the ocean as a whole) because of the great wavelength involved. Shallow water permits tsunamis to increase in amplitude without significant reduction in velocity and energy. On the other hand, where the water is relatively deep off a shoreline, the growth in size of the wave is restricted. Large waves, several metres in height, are most likely when tsunamis move into narrowing inlets.

Usually, the first wave is like a very rapid change in tide. For example, the sea level may change 7 or 8 m in 10 min. A bore occurs where there is a concentration of wave energy by funnelling, as in bays, or by convergence, as on points. A steep front rolls over relatively quiet water. Behind the front, the crest of such a wave is broad and flat, and the wave velocity is about 30 km h^{-1}. Along rocky coasts, large blocks of material may be dislodged and moved shoreward.

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Chapter 4

Groundwater Conditions and Supply

The Origin and Occurrence of Groundwater

The principal source of groundwater is meteoric water, that is, precipitation. However, two other sources are occasionally of some consequence. These are juvenile water and connate water. The former is derived from magmatic sources, whereas the latter represents the water in which sediments are deposited. Connate water is trapped in the pore spaces of sedimentary rocks as they are formed and has not been expelled.

The amount of water that infiltrates into the ground depends on how precipitation is dispersed, namely, on the proportions that are assigned to immediate run-off and to evapotranspiration, the remainder constituting the proportion allotted to infiltration/percolation. Infiltration refers to the seepage of surface water into the ground, percolation being its subsequent movement, under the influence of gravity, to the zone of saturation. In reality, one cannot be separated from the other. The infiltration capacity is influenced by the rate at which rainfall occurs (which also affects the quantity of water available), the vegetation cover, the porosity of the soils and rocks, their initial moisture content and the position of the zone of saturation.

The retention of water in soil depends on the capillary force and the molecular attraction of the particles. As the pores in the soil become thoroughly wetted, the capillary force declines, so that gravity becomes more effective. In this way, downward percolation can continue after infiltration has ceased but the capillarity increases in importance as the soil dries. No further percolation occurs after the capillary and gravity forces are balanced. Thus, water percolates into the zone of saturation when the retention capacity is satisfied. This means that the rains that occur after the deficiency of soil moisture has been catered for are those that count as far as supplementing groundwater is concerned.

The Water Table or Phreatic Surface

The pores within the zone of saturation are filled with water, generally referred to as phreatic water. The upper surface of this zone is therefore known as the phreatic surface but is more commonly termed the water table. Above the zone of saturation is the zone of aeration in which both air and water occupy the pores. The water in the zone of aeration is referred to as vadose water. This zone is divided into three subzones, those of soil water, the intermediate belt and the capillary fringe. The uppermost or soil water belt discharges water into the atmosphere in perceptible quantities by evapotranspiration. In the capillary fringe, which occurs immediately above the water table, water is held in the pores by capillary action. An immediate belt occurs when the water table is far enough below the surface for the soil water belt not to extend down to the capillary fringe. The degree of saturation decreases from the water table upwards.

The geological factors that influence percolation not only vary from one recharge area to another but may do so within the same one. This, together with the fact that rain does not fall evenly over a given area, means that the contribution to the zone of saturation is variable. This, in turn, influences the position of the water table, as do the points of discharge. A rise in the water table as a response to percolation is controlled partly by the rate at which water can drain from the area of recharge. Accordingly, it tends to be greatest in areas of low transmissivity (see below). Mounds and ridges form in the water table under the areas of greatest recharge. Superimpose on these the influence of water draining from lakes and streams, and it can be appreciated that a water table is continually adjusting towards equilibrium. Because of the low flow rates in most rocks, this equilibrium is rarely, if ever, attained before another disturbance occurs. By using measurements of groundwater levels obtained from wells and by observing the levels at which springs discharge, it is possible to construct groundwater contour maps showing the form and elevation of the water table (Fig. 4.1).

The water table fluctuates in position, particularly in those climates where there are marked seasonal changes in rainfall. Hence, permanent and intermittent water tables can be distinguished, the former marking the level beneath which the water table does not sink, whereas the latter is an expression of the fluctuation. Usually, water tables fluctuate within the lower and upper limits rather than between them, this is especially the case in humid regions since the periods between successive recharges are small. The position at which the water table intersects the surface is termed the spring line. Intermittent and permanent springs similarly can be distinguished.

A perched water table is one that forms above a discontinuous impermeable layer such as a lens of clay in a formation of sand, the clay impounding a water mound.

Aquifers, Aquicludes and Aquitards

An aquifer is the term given to a rock or soil mass that not only contains water but from which water can be abstracted readily in significant quantities. The ability of an aquifer to transmit

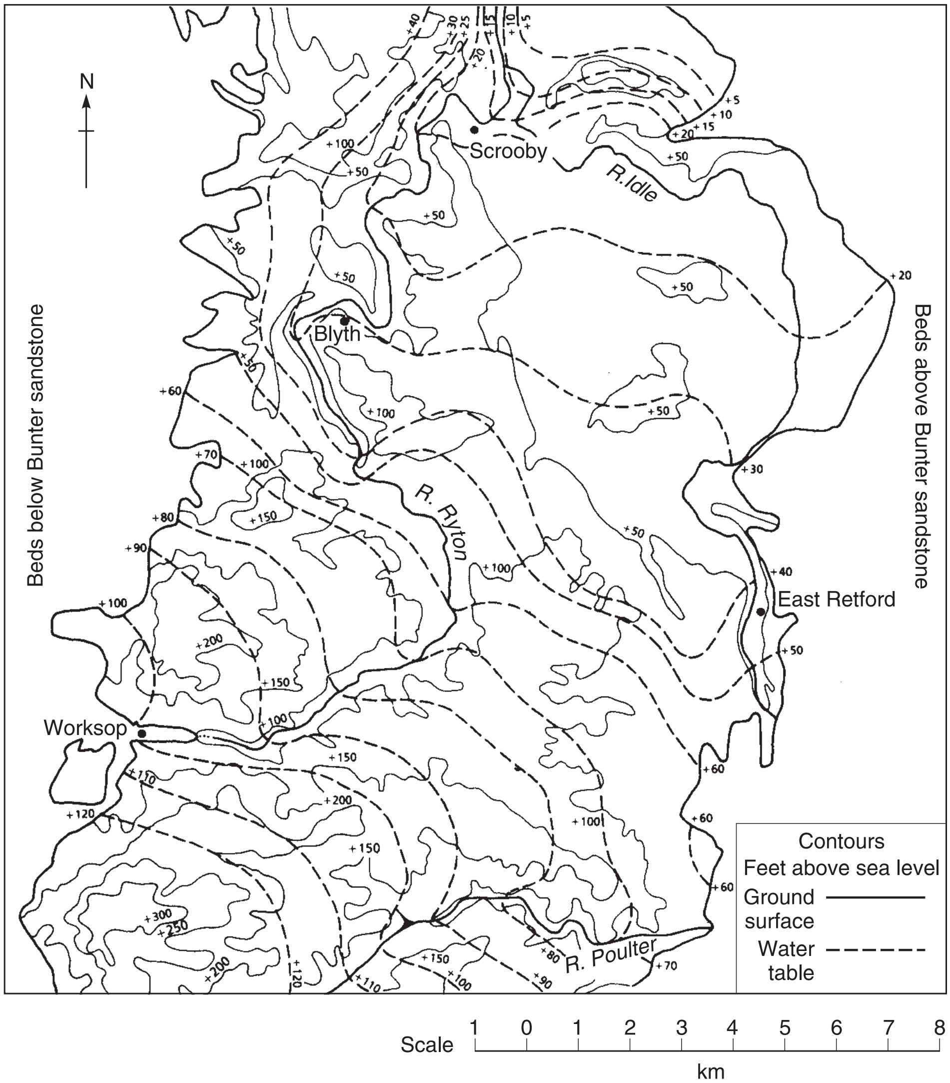


Figure 4.1

Map of part of Nottinghamshire showing the water table in the Bunter Sandstone (now the Sherwood Sandstone).

water is governed by its permeability. Indeed, the permeability of an aquifer usually is in excess of 10^{-5} m s^{-1}.

By contrast, a formation with a permeability of less than 10^{-9} m s^{-1} is one that, in engineering terms, is regarded as impermeable and is referred to as an aquiclude. For example, clays and shales are aquicludes. Even when such rocks are saturated, they tend to impede the flow of water through stratal sequences.

An aquitard is a formation that transmits water at a very slow rate but that, over a large area of contact, may permit the passage of large amounts of water between adjacent aquifers that it separates. Sandy clays provide an example.

An aquifer is described as unconfined when the water table is open to the atmosphere, that is, the aquifer is not overlain by material of lower permeability (Fig. 4.2a). Conversely, a confined aquifer is one that is overlain by impermeable rocks (Fig. 4.2a). Confined aquifers may have relatively small recharge areas as compared with unconfined aquifers and, therefore, may yield less water. A leaky aquifer is one which is overlain and/or underlain by aquitard(s) (Fig. 4.2b).

Very often, the water in a confined aquifer is under piezometric pressure, that is, there is an excess of pressure sufficient to raise the groundwater above the base of the overlying bed when the aquifer is penetrated by a well. Piezometric pressures are developed when the

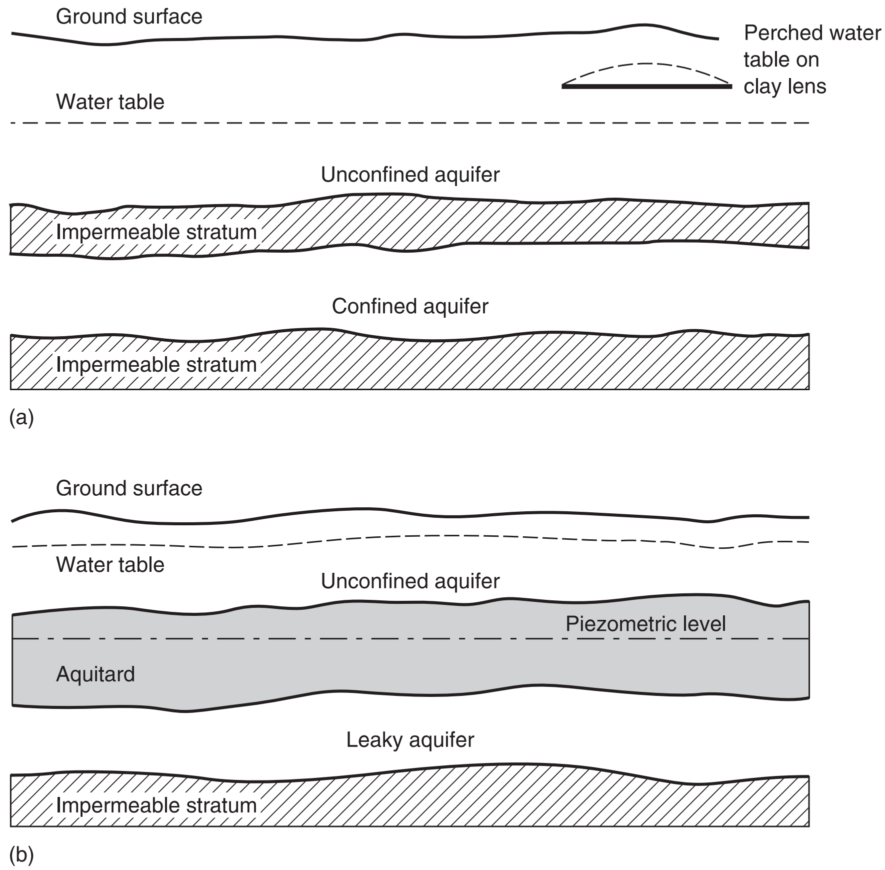


Figure 4.2

(a) Diagram illustrating unconfined and confined aquifers with a perched water table in the vadose zone. (b) Diagram illustrating a leaky aquifer.

buried upper surface of a confined aquifer is lower than the water table in the aquifer at its recharge area. Where the piezometric surface is above ground level, the water overflows from a well. Such wells are described as artesian. A synclinal structure is the commonest cause of artesian conditions (Fig. 4.3a). Other geological structures that give rise to artesian conditions are illustrated in Figure 4.3b. The term subartesian is used to describe those

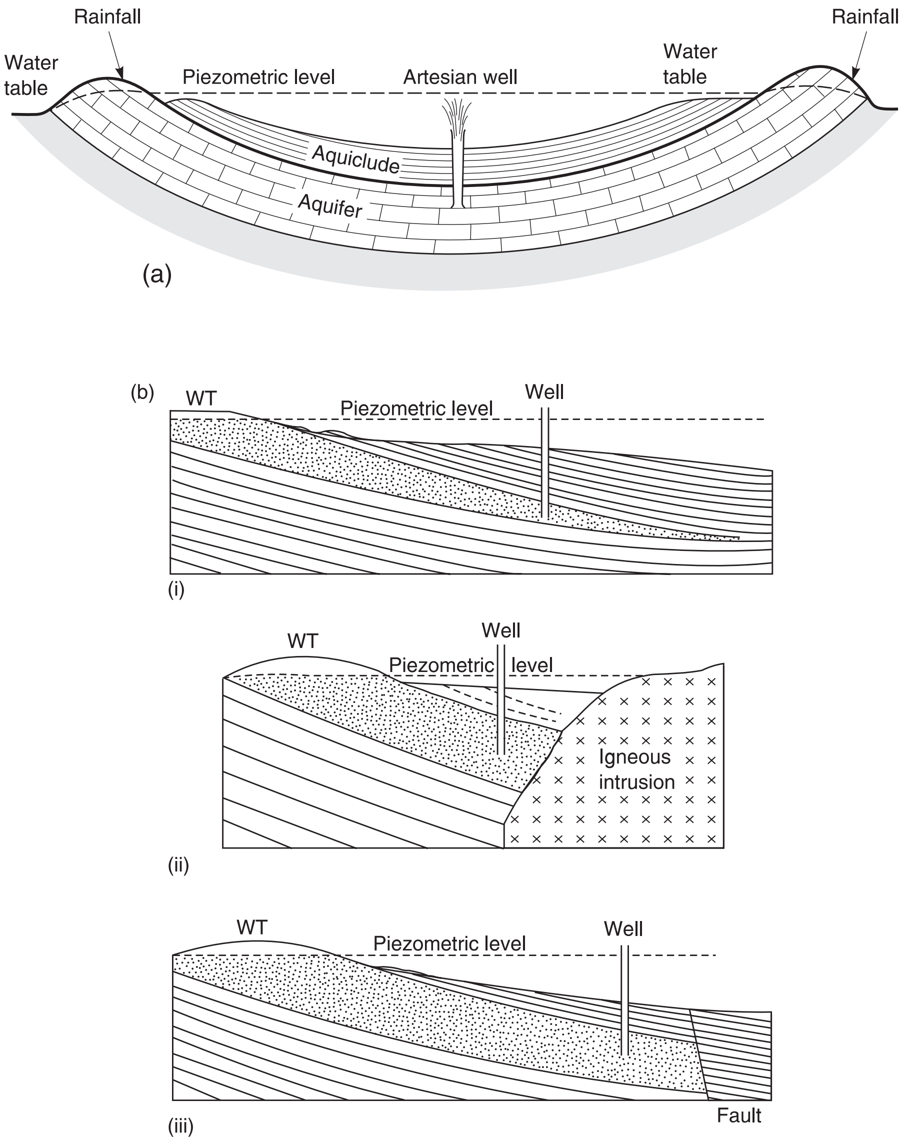


Figure 4.3

(a) Section across an artesian basin; (b) other examples of artesian conditions (permeable layer, stippled, sandwiched between impermeable beds).

conditions in which the groundwater is not under sufficient piezometric pressure to rise to the ground surface.

Capillary Movement in Soil

Capillary movement in soil refers to the movement of moisture through the minute pores between the soil particles that act as capillaries. It takes place as a consequence of surface tension, therefore moisture can rise from the water table. This movement, however, can occur in any direction, not just vertically upwards. It occurs whenever evaporation takes place from the surface of the soil, thereby exerting a “surface tension pull” on the moisture, the forces of surface tension increasing as evaporation proceeds. Accordingly, capillary moisture is in hydraulic continuity with the water table and is raised against the force of gravity. Equilibrium is attained when the forces of gravity and surface tension are balanced.

The boundary separating capillary moisture from the gravitational groundwater in the zone of saturation is ill-defined and cannot be determined accurately. The zone immediately above the water table that is saturated with capillary moisture is referred to as the closed capillary fringe, whereas above this, air and capillary moisture exist together in the pores of the open capillary fringe. The height of the capillary fringe depends largely on the particle size distribution and density of the soil mass that, in turn, influence pore size. In other words, the smaller the pore size, the greater is the height of the fringe. For example, capillary moisture can rise to great heights in clay soils but the movement is very slow (Table 4.1). The height of the capillary fringe in soils that are poorly graded generally varies, whereas in uniformly textured soils, it attains roughly the same height. Where the water table is at shallow depth and the maximum capillary rise is large, moisture is continually attracted from the water table, due to evaporation from the surface, so that the uppermost soil is near saturation.

Below the water table, the groundwater contained in the pores is under normal hydrostatic load, the pressure increasing with depth. Because these pressures exceed atmospheric pressure,

Table 4.1. Capillary rises and pressures in soils

Soil	Capillary rise (mm)	Capillary pressure (kPa)
Fine gravel	Up to 100	Up to 1.0
Coarse sand	100–150	1.0–1.5
Medium sand	150–300	1.5–3.0
Fine sand	300–1000	3.0–10.0
Silt	1000–10,000	10.0–100.0
Clay	Over 10,000	Over 100.0

Table 4.2. Soil suction pressure and pF value

pF value	(mm water)	Equivalent suction (kPa)
0	10	0.1
1	100	1.0
2	1000	10.0
3	10,000	100.0
4	100,000	1,000.0
5	1,000,000	10,000.0

they are designated positive pressures. On the other hand, the pressures existing in the capillary zone are less than atmospheric, and so are termed negative pressures. Therefore, the water table usually is regarded as a datum of zero pressure between the positive pore water pressure below and the negative above.

At each point where moisture menisci are in contact with soil particles, the forces of surface tension are responsible for the development of capillary or suction pressure (Table 4.1). The air and groundwater interfaces move into the smaller pores. In so doing, the radii of curvature of the interfaces decrease, and the soil suction increases. Hence, the drier the soil, the higher is the soil suction.

Soil suction is a negative pressure and indicates the height to which a column of water could rise due to such suction. Since this height or pressure may be very large, a logarithmic scale has been adopted to express the relationship between soil suction and moisture content; this is referred to as the pF value (Table 4.2).

Soil suction tends to force soil particles together, and these compressive stresses contribute towards the strength and stability of the soil. There is a particular suction pressure for a particular moisture content in a given soil, the magnitude of which is governed by whether it is becoming wetter or drier. In fact, as clay soil dries out, the soil suction may increase to the order of several thousand kilopascals. However, the strength of a soil attributable to soil suction is only temporary and is destroyed upon saturation. At that point, soil suction is zero.

Porosity and Permeability

Porosity and permeability are the two most important factors governing the accumulation, migration and distribution of groundwater. However, both may change within a rock or soil mass in the course of its geological evolution. Furthermore, it is not uncommon to find variations in both porosity and permeability per metre of depth beneath the ground surface.

Porosity

The porosity, n, of a rock can be defined as the percentage pore space within a given volume and is expressed as follows:

$$n = (V_v / V) \times 100 \tag{4.1}$$

where V_v is the volume of the voids and V is the total volume of the material concerned. A closely related property is the void ratio, e, that is, the ratio of the volume of the voids to the volume of the solids, V_s:

$$e = V_v / V_s \tag{4.2}$$

Where the ground is saturated, the void ratio can be derived from:

$$e = mG_s \tag{4.3}$$

m being the moisture content and G_s the specific gravity. Both the porosity and the void ratio indicate the relative proportion of void volume in the material, and the relationships between the two are as follows:

$$n = e/(1 + e) \tag{4.4}$$

and

$$e = n/(1 - n) \tag{4.5}$$

Total or absolute porosity is a measure of the total void volume and is the excess of bulk volume over grain volume per unit of bulk volume. It is usually determined as the excess of grain density (i.e. specific gravity) over dry density per unit of grain density and can be obtained from the following expression:

$$\text{Absolute porosity} = \left(1 - \frac{\text{Dry density}}{\text{Grain density}}\right) \times 100 \tag{4.6}$$

The effective, apparent or net porosity is a measure of the effective void volume of a porous medium and is determined as the excess of bulk volume over grain volume and occluded pore volume. It may be regarded as the pore space from which water can be removed. Groundwater does not drain from occluded pores.

The factors affecting the porosity of soil include particle size distribution, sorting, grain shape, fabric, degree of compaction, solution effects and, lastly, mineralogical composition, particularly the presence of clay particles (Bell, 1978). The highest porosity is attained when all the grains are of the same size. The addition of grains of different sizes to such an assemblage lowers its porosity and this, within certain limits, is directly proportional to the amount added. Irregularities in grain shape result in a larger possible range of porosity, as irregular forms may theoretically be packed either more tightly or more loosely than spheres. Similarly, angular grains may either cause an increase or a decrease in porosity. After a sediment has been buried and indurated, several additional factors help determine its porosity. The chief among these are closer spacing of grains, deformation and granulation of grains, recrystallization, secondary growth of minerals, cementation and, in some cases, dissolution. Hence, the diagenetic changes undergone by a sedimentary rock may either increase or decrease its original porosity.

The porosity can be determined experimentally by using either the standard saturation method or an air porosimeter. Both tests give an effective value of porosity, although that obtained by the air porosimeter may be somewhat higher because air can penetrate pores more easily than can water.

The porosity of a deposit does not necessarily provide an indication of the amount of water that can be obtained therefrom. Nevertheless, the water content of a soil or rock is related to its porosity. The water content, m, of a porous material usually is expressed as the percentage of the weight of the solid material, W_s, that is:

$$m = (W_w/W_s) \times 100, \tag{4.7}$$

where W_w is the weight of the water. The degree of saturation, S_r, refers to the relative volume of water, V_w, in the voids, V_v, and is expressed as a percentage:

$$S_r = (V_w/V_v) \times 100 \tag{4.8}$$

Specific Retention and Specific Yield

As far as supply is concerned, the capacity of a material to yield water is of greater importance than its capacity to hold water. Even though a rock or soil may be saturated, only a certain proportion of water can be removed by drainage under gravity or pumping, the remainder being held in place by capillary or molecular forces. The ratio of the volume of water retained, V_{wr}, to that of the total volume of rock or soil, V, expressed as a percentage, is referred to as the specific retention, S_{re}:

$$S_{re} = (V_{wr}/ V) \times 100 \tag{4.9}$$

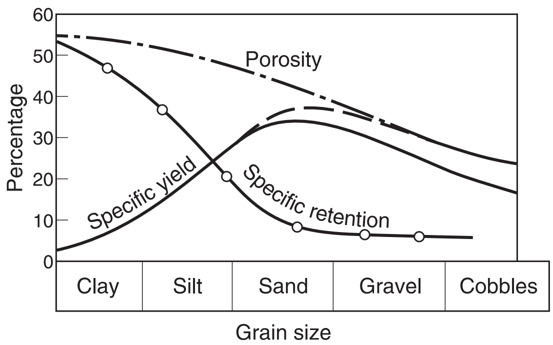


Figure 4.4

Relationship between grain size, porosity, specific retention and specific yield. Well-sorted material (— - —), average material (——).

The amount of water retained varies directly in accordance with the surface area of the pores and indirectly with regard to the pore space. The specific surface of a particle is governed by its size and shape. For example, particles of clay have far larger specific surfaces than do grains of sand. As an illustration, a grain of sand, 1 mm in diameter, has a specific surface of about 0.002 $m^2 g^{-1}$, compared with kaolinite, which varies from approximately 10 to 20 $m^2 g^{-1}$. Hence, clays have a much higher specific retention than sands (Fig. 4.4).

The specific yield, S_y, of a rock or soil mass refers to its water-yielding capacity, attributable to gravity drainage as occurs when the water table declines. It is the ratio of the volume of water, after saturation, that can be drained by gravity, V_{wd}, to the total volume of the aquifer, expressed in percentage as:

$$S_y = (V_{wd} / V) \times 100 \tag{4.10}$$

The specific yield plus the specific retention is equal to the porosity of the material:

$$n = S_y + S_{re}, \tag{4.11}$$

when all the pores are interconnected. The relationship between the specific yield and particle size distribution is shown in Figure 4.4. In soils, the specific yield tends to decrease as the coefficient of uniformity increases. The coefficient of uniformity is the ratio between D_{60} and D_{10}, where D_{60} and D_{10} are the sizes at which 60% and 10% of the particles are smaller, respectively. Examples of the specific yield of some common types of soil and rock are given in Table 4.3 (it must be appreciated that individual values of specific yield can vary from those quoted).

Table 4.3. Some examples of specific yield

Material	Specific yield (%)
Gravel	15–30
Sand	10–30
Dune sand	25–35
Sand and gravel	15–25
Loess	15–20
Silt	5–10
Clay	1–5
Till (silty)	4–7
Till (sandy)	12–18
Sandstone	5–25
Limestone	0.5–10
Shale	0.5–5

Permeability

Permeability may be defined as the ability of soil or rock to allow the passage of fluids into or through it without impairing its structure. In ordinary hydraulic usage, a substance is termed permeable when it permits the passage of a measurable quantity of fluid in a finite period of time, and impermeable when the rate at which it transmits that fluid is slow enough to be negligible under existing temperature–pressure conditions (Table 4.4). The permeability of a particular material is defined by its coefficient of permeability or hydraulic conductivity, k. The transmissivity or flow in m^3 day^{-1} through a section of aquifer 1 m wide under a hydraulic gradient of unity is sometimes used as a convenient quantity in the calculation of groundwater flow instead of the coefficient of permeability. The transmissivity, T, and coefficient of permeability, k, are related to each other as follows:

$$T = kH, \tag{4.12}$$

where H is the saturated thickness of the aquifer.

The flow through a unit cross section of material is modified by temperature, hydraulic gradient and the coefficient of permeability. The latter is affected by the uniformity and range of grain size, shape of the grains, stratification, the amount of consolidation and cementation undergone, and the presence and nature of discontinuities. Temperature changes affect the flow rate of a fluid by changing its viscosity. The rate of flow commonly is assumed to be directly proportional to the hydraulic gradient but this is not always so in practice.

Table 4.4. Relative values of permeabilities

Rock types	Porosity: Primary (grain)%	Porosity: Secondary (fracture)*	Permeability range (m s^{-1}): 10^{0} 10^{-2} 10^{-4} 10^{-6} 10^{-8} 10^{-10}; Very High, High, Medium, Low, Very low, Impermeable	Well yields: High, Medium, Low	Type of water-bearing unit
Sediments, unconsolidated					
Gravel	30–40				Aquifer
Coarse sand	30–40				Aquifer
Medium to fine sand	25–35				Aquifer
Silt	40–50	Occasional			Aquiclude
Clay, till	45–55	Often fissured			Aquiclude
Sediments, consolidated					
Limestone, dolostone	1–50	Solution joints bedding planes			Aquifer or aquiclude
Coarse, medium sandstone	<20	Joints and bedding planes			Aquifer or aquiclude
Fine sandstone	<10	Joints and bedding planes			Aquifer or aquiclude
Shale, Siltstone	–	Joints and bedding planes			Aquiclude or aquifer
Volcanic rocks, e.g. basalt	–	Joints and "bedding" planes			Aquifer or aquiclude
Plutonic and metamorphic rocks		Weathering and joints decreasing as depth increases			Aquiclude or aquifer

*Rarely exceeds 10%

Permeability and porosity are not necessarily as closely related as would be expected. For instance, very-fine-textured sandstones may have a higher porosity than coarser ones, though the latter are more permeable.

As can be inferred from above, the permeability of a clastic material also is affected by the interconnections between the pore spaces. If these are highly tortuous, then the permeability is reduced accordingly. Consequently, tortuosity figures importantly in permeability, influencing the extent and rate of free-water saturation. It can be defined as the ratio of the total path covered by a liquid flowing in the pore channels between two given points to the straight line distance between them. The sizes of the throat areas between pores obviously are important.

Stratification in a formation varies within limits both vertically and horizontally. It frequently is difficult to predict what effect stratification has on the permeability of the beds. Nevertheless, in the great majority of cases where a directional difference in permeability exists, the greater permeability is parallel to the bedding. Ratios of 5:1 are not uncommon in sandstones.

The permeability of intact rock (primary permeability) is usually several orders less than in situ permeability (secondary permeability). Hence, as far as the assessment of flow through rock masses is concerned, field tests provide more reliable results than can be obtained from testing intact samples in the laboratory. Although secondary permeability is affected by the frequency, continuity and openness, and amount of infilling of discontinuities, a rough estimate of the permeability can be obtained from their frequency (Table 4.5). Admittedly, such estimates must be treated with caution and cannot be applied to rocks that are susceptible to solution.

Dykes often act as barriers to groundwater flow so that the water table on one side may be higher than on the other. Fault planes occupied by clay gouge may have a similar effect. Conversely, a fault plane may act as a conduit where it is not sealed.

Table 4.5. Estimation of secondary permeability from discontinuity frequency

Term	**Interval (m)**	**Permeability rock mass description**	**Coefficient of permeability *k* (m s^{-1})**
Very closely to extremely closely spaced discontinuities	Less than 0.2	Highly permeable	10^{-2}–1
Closely to moderately widely spaced discontinuities	0.2–0.6	Moderately permeable	10^{-5}–10^{-2}
Widely to very widely spaced discontinuities	0.6–2.0	Slightly permeable	10^{-9}–10^{-5}
No discontinuities	Over 2.0	Effectively impermeable	Less than 10^{-9}

Storage Coefficient

The storage coefficient or storativity, S, of an aquifer has been defined as the volume of water released from or taken into storage per unit surface area of the aquifer, per unit change in head normal to that surface (Fig. 4.5). It is a dimensionless quantity. Changes in storage in an unconfined aquifer represent the product of the volume of the aquifer, between the water table before and after a given period of time, and the specific yield. Indeed, the storage coefficient of an unconfined aquifer virtually corresponds to the specific yield as more or less all the groundwater is released from storage by gravity drainage and only an extremely small part results from compression of the aquifer and the expansion of water.

However, in confined aquifers, water is not yielded simply by gravity drainage from the pore space because there is no falling water table and the material remains saturated. Hence, other factors are involved regarding yield, such as consolidation of the aquifer and expansion of groundwater consequent upon lowering of the piezometric surface. Therefore, much less water is yielded by confined than unconfined aquifers.

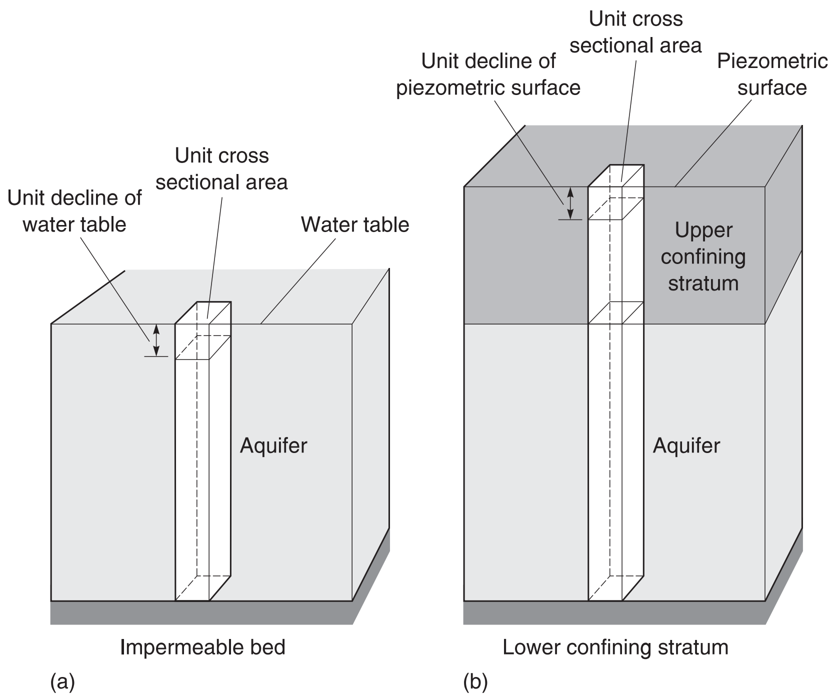


Figure 4.5

Diagram illustrating the storage coefficient of (a) an unconfined aquifer and (b) a confined aquifer.

Flow through Soils and Rocks

Water possesses three forms of energy, namely, potential energy attributable to its height, pressure energy owing to its pressure, and kinetic energy due to its velocity. The latter can usually be discounted in any assessment of flow through soils. Energy in water usually is expressed in terms of head. The head possessed by groundwater in soils or rocks is manifested by the height to which it will rise in a standpipe above a given datum. This height usually is referred to as the piezometric level and provides a measure of the total energy of the water. If at two different points within a continuous area of groundwater, there are different amounts of energy, then there will be a flow towards the point of lesser energy and the difference in head is expended in maintaining that flow. Other things being equal, the velocity of flow between two points is directly proportional to the difference in head between them. The hydraulic gradient, i, refers to the loss of head or energy of water flowing through the ground. This loss of energy by the groundwater is due to the friction resistance of the ground material, and this is greater in fine- than coarse-grained soils. Thus, there is no guarantee that the rate of flow will be uniform, indeed this is exceptional. However, if it is assumed that the resistance to flow is constant, then for a given difference in head, the flow velocity is directly proportional to the flow path.

Darcy's Law

Before any mathematical treatment of groundwater flow can be attempted, certain simplifying assumptions have to be made, namely, that the material is isotropic and homogeneous, that there is no capillary action and that a steady state of flow exists. Since rocks and soils are anisotropic and heterogeneous, as they may be subject to capillary action and flow through them is characteristically unsteady, any mathematical assessment of flow must be treated with caution.

The basic law concerned with flow is that enunciated by Darcy (1856), which states that the rate of flow, v, per unit area is proportional to the gradient of the potential head, i, measured in the direction of flow, k being the coefficient of permeability:

$$v = ki \tag{4.13}$$

and for a particular rock or soil or part of it of area A:

$$Q = vA = Aki \tag{4.14}$$

where Q is the quantity in a given time. The ratio of the cross-sectional area of the pore spaces in a soil to that of the whole soil is given by $e/(1 + e)$, where e is the void ratio. Hence, a truer velocity of flow, that is, the seepage velocity, v_s, is

$$v_s = [(1 + e)/e]ki \tag{4.15}$$

Darcy's law is valid as long as a laminar flow exists. Departures from Darcy's law therefore occur when the flow is turbulent, such as when the velocity of flow is high. Such conditions exist in very permeable media. Accordingly, it usually is accepted that this law can be applied to those soils that have finer textures than gravels. Furthermore, Darcy's law probably does not accurately represent the flow of water through a porous medium of extremely low permeability because of the influence of surface and ionic phenomena, and the presence of any gases.

Apart from an increase in the mean velocity, the other factors that cause deviations from the linear laws of flow include the non-uniformity of pore spaces, since differing porosity gives rise to differences in the seepage rates through pore channels. A second factor is the absence of a running-in section where the velocity profile can establish a steady state parabolic distribution. Lastly, such deviations may be developed by perturbations due to jet separation from wall irregularities.

Darcy failed to recognize that permeability also depends on the density, ρ, and dynamic viscosity of the fluid involved, μ, and the average size, D_n, and shape of the pores in a porous medium. In fact, permeability is directly proportional to the unit weight of the fluid concerned and is inversely proportional to its viscosity. The latter is influenced very much by temperature. The following expression attempts to take these factors into account:

$$k = CD_n^2\rho/\mu \qquad (4.16)$$

where C is a dimensionless constant or shape factor that takes note of the effects of stratification, packing, particle size distribution and porosity. It is assumed in this expression that both the porous medium and the water are mechanically and physically stable, but this may never be true. For example, ionic exchange on clay and colloid surfaces may bring about changes in mineral volume that, in turn, affect the shape and size of the pores. Moderate to high groundwater velocities tend to move colloids and clay particles. Solution and deposition may result from the pore fluids. Small changes in temperature and/or pressure may cause gas to come out of solution that may block pore spaces.

The Kozeny–Carmen equation for deriving the coefficient of permeability also takes the porosity, n, into account as well as the specific surface area of the porous medium, S_a, that is defined per unit volume of solid

$$k = C_o \frac{n^3}{(1-n)^2 S_a^{\,2}} \qquad (4.17)$$

where C_o is a coefficient, the suggested value of which is 0.2.

General Equation of Flow

When considering the general case of flow in porous media, it is assumed that the media is isotropic and homogeneous as far as permeability is concerned. If an element of saturated material is taken, with the dimensions dx, dy and dz (Fig. 4.6), and flow is taking place in the x–y plane, then the generalized form of Darcy's Law is:

$$v_x = k_x i_x \tag{4.18}$$

$$v_x = k_x \left(\frac{\delta h}{\delta x}\right) \tag{4.19}$$

and

$$v_y = k_y i_y \tag{4.20}$$

$$v_y = k_y \left(\frac{\delta h}{\delta y}\right) \tag{4.21}$$

where h is the total head under steady state conditions and k_x, i_x and k_y, i_y are, respectively, the coefficients of permeability and the hydraulic gradients in the x and y directions. Assuming that the fabric of the medium does not change and that the water is incompressible, the volume of water entering the element is the same as that leaving in any given time, hence:

$$v_x \mathrm{d}y\mathrm{d}z + v_y \mathrm{d}x\mathrm{d}z = \left(v_x + \frac{\delta v_x}{\delta x}\mathrm{d}x\right)\mathrm{d}y\mathrm{d}z + \left(v_y + \frac{\delta v_y}{\delta y}\mathrm{d}y\right)\mathrm{d}x\mathrm{d}z \tag{4.22}$$

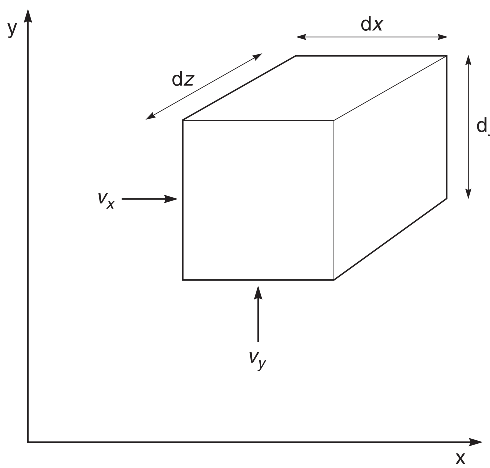


Figure 4.6

Seepage through an element of soil.

In such a situation, the difference in volume between the water entering and leaving the element is zero; therefore:

$$\frac{\delta v_x}{\delta x} + \frac{\delta v_y}{\delta y} = 0 \tag{4.23}$$

Equation 4.23 is referred to as the flow continuity equation. If Eqs. 4.19 and 4.21 are substituted in the continuity equation, then:

$$k_x\left(\frac{\delta^2 h}{\delta x^2}\right) + k_y\left(\frac{\delta^2 h}{\delta y^2}\right) = 0 \tag{4.24}$$

If there is a recharge or discharge to the aquifer (−w and +w, respectively), then this term must be added to the right-hand side of Eq. 4.24. If it is assumed that the coefficient of permeability is isotropic throughout the media so that $k_x = k_y$, then Eq. 4.24 becomes:

$$\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} = 0 \tag{4.25}$$

This is the two-dimensional Laplace equation for steady state flow in an isotropic porous medium. The partial differential equation governing the two-dimensional unsteady flow of water in an anisotropic aquifer can be written as:

$$T_x = \frac{\delta^2 h}{\delta x^2} + T_y \frac{\delta^2 h}{\delta y^2} = S\frac{\delta h}{\delta t} \tag{4.26}$$

where T and S are the coefficients of transmissivity and storage, respectively.

Flow through Stratified Deposits

In a stratified sequence of deposits, the individual beds, no doubt, will have different permeabilities, so that vertical permeability will differ from horizontal permeability. Consequently, in such situations, it may be necessary to determine the average values of the coefficient of permeability normal to, k_v, and parallel to, k_h, the bedding. If the total thickness of the sequence is H_T and the thickness of the individual layers are H_1, H_2, H_3, ..., H_n, with the corresponding values of the coefficient of permeability k_1, k_2, k_3, ..., k_n, then k_v and k_h can be obtained as follows:

$$k_v = \frac{H_T}{H_1/k_1 + H_2/k_2 + H_3/k_3 + \cdots + H_n/k_n} \tag{4.27}$$

and

$$k_h = \frac{H_1 / k_1 + H_2 / k_2 + H_3 / k_3 + \cdots + H_n / k_n}{H_T} \tag{4.28}$$

Fissure Flow

Generally, it is the interconnected systems of discontinuities that determine the permeablility of a particular rock mass. Indeed, the permeability of a jointed rock mass is usually several orders higher than that of intact rock. According to Serafim (1968), the following expression can be used to derive the filtration through a rock mass intersected by a system of parallel-sided joints with a given opening, *e*, separated by a given distance, *d*:

$$k = \frac{e^3 \gamma_w}{12 d \mu} \tag{4.29}$$

where γ_w is the unit weight of water and μ its viscosity. The velocity of flow, *v*, through a single joint of constant gap, *e*, is expressed by:

$$v = \left(\frac{e^2 \gamma_w}{12 \mu} \right) i \tag{4.30}$$

where *i* is the hydraulic gradient.

Pore Pressures, Total Pressures and Effective Pressures

Subsurface water is normally under pressure, which increases with increasing depth below the water table to very high values. Such water pressures have a significant influence on the engineering behaviour of soil masses, and their variations are responsible for changes in the stresses in these masses, which affect their deformation characteristics and failure.

The efficiency of a soil in supporting a structure is influenced by the effective or intergranular pressure, that is, the pressure between the particles of the soil that develops resistance to applied load. Because the moisture in the pores offers no resistance to shear, it is neutral and therefore pore water pressure also has been referred to as neutral pressure. Since the pore water or neutral pressure plus the effective pressure equals the total pressure, reduction in pore water pressure increases the effective pressure. Reduction of the pore water pressure by drainage consequently affords better conditions for carrying a proposed structure.

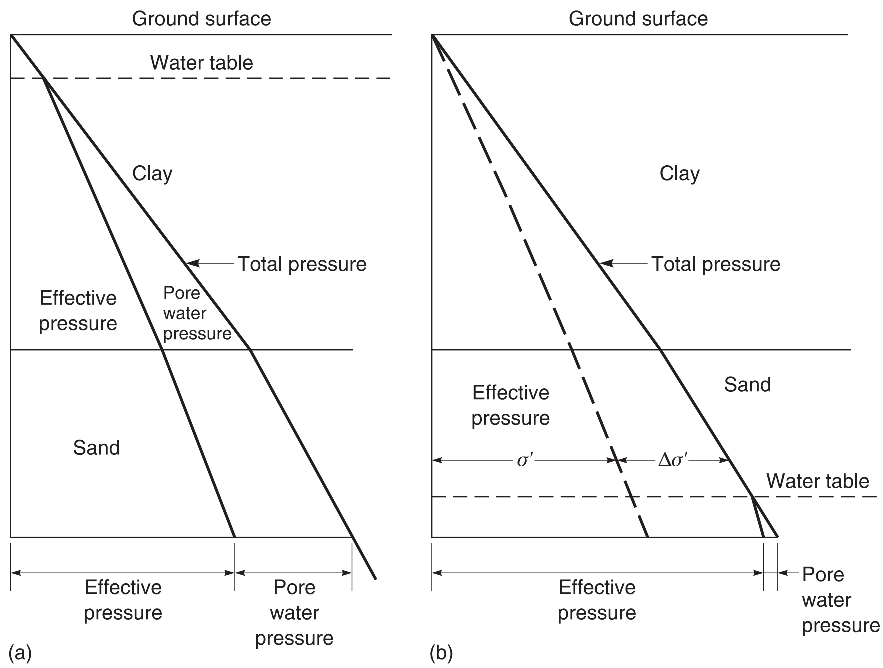


Figure 4.7

Pressure diagrams to illustrate the influence of lowering the water table on effective pressure: (a) the water tabel is just below the ground surface, (b) the water table has been lowered into the sand and the effective pressure, σ', is increased accordingly. In the clay, the effective pressure and total pressure, σ, are the same.

The effective pressure at a particular depth is obtained by multiplying the unit weight of the soil by the depth in question and subtracting the pore water pressure for that depth. In a layered sequence, the individual layers may have different unit weights. The unit weight of each layer should be multiplied by its thickness, and the involved pore water pressure should be subtracted. The effective pressure for the total thickness involved is obtained by summing the effective pressures of the individual layers (Fig. 4.7). Water held in the capillary fringe by soil suction does not affect the values of pore pressure below the water table. However, the weight of water held in the capillary fringe does increase the weight of overburden and so the effective pressure.

Piezometers are installed in the ground in order to monitor and obtain measurements of pore water pressures (Fig. 4.8). Observations should be made regularly so that changes due to external factors such as excessive precipitation, tides, the seasons, etc., are noted, it being important to record the maximum pressures that have occurred. Standpipe piezometers allow the determination of the position of the water table and the permeability. For example, the water level can be measured with an electric dipmeter. Piezometer tips that have leads going to a constant head permeability unit, enable the rate of flow through the tip to

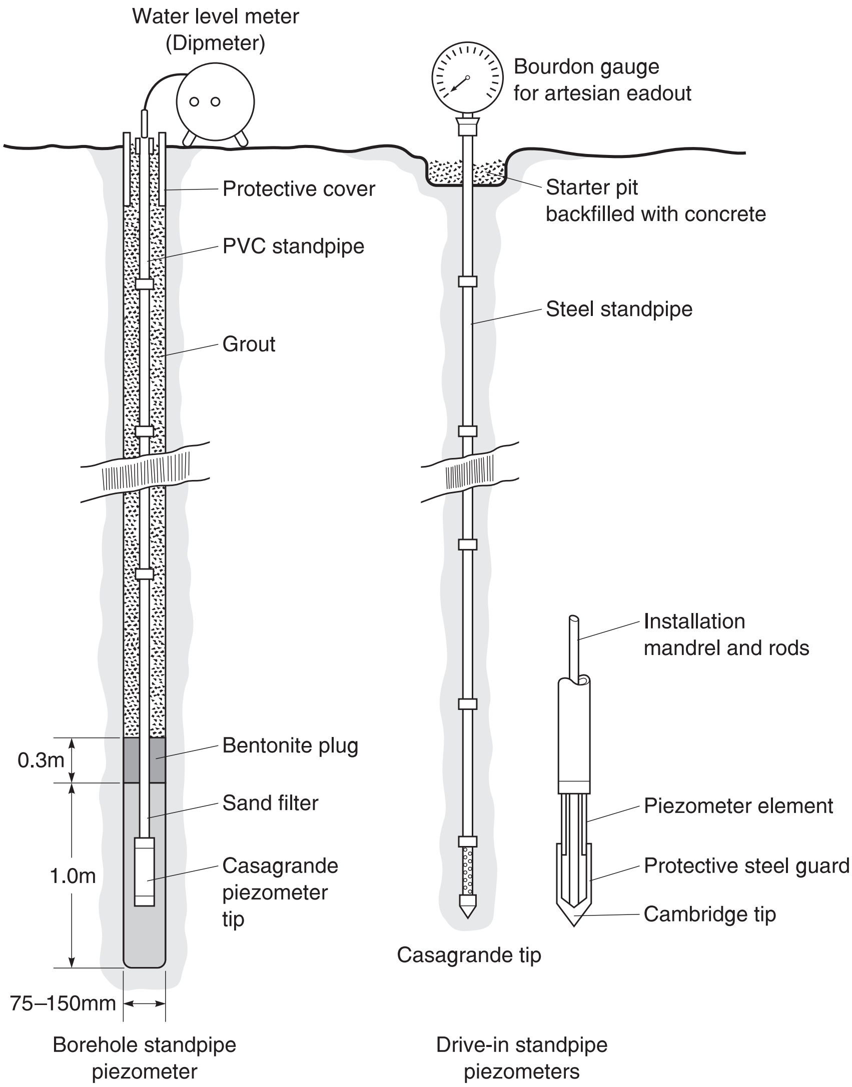


Figure 4.8

Standard piezometers: (a) a borehole standpipe piezometer and (b) a drive-in standpipe piezometer.

be measured. Hydraulic piezometers can be installed at various depths in a borehole where it is required to determine water pressures. They are connected to a manometer board that records the changes in pore water pressure. Usually, simpler types of piezometer are used in the more permeable soils. When a piezometer is installed in a borehole, it should be surrounded with a filter of clean sand. The sand should be sealed both above and below the piezometer to enable the pore water pressures at that particular level to be measured. The response to piezometers in rock masses can be influenced very much by the incidence and geometry of the discontinuities, therefore, the original values of water pressure obtained may be misleading if due regard is not given to these structures.

Critical Hydraulic Gradient, Quick Conditions and Hydraulic Uplift Phenomena

As water flows through the soil and loses head, its energy is transferred to the particles past which it is moving, which in turn creates a drag effect on the particles. If the drag effect is in the same direction as the force of gravity, then the effective pressure is increased and the soil is stable. Indeed, the soil tends to become more dense. Conversely, if water flows towards the surface, then the drag effect counters gravity, thereby reducing the effective pressure between particles. If the velocity of upward flow is sufficient, it can buoy up the particles so that the effective pressure is reduced to zero. This represents a critical condition in which the weight of the submerged soil is balanced by the upward-acting seepage force. The critical hydraulic gradient, i_c, can be calculated from the following expression:

$$i_c = \frac{G_s - 1}{1 + e}, \qquad (4.31)$$

where G_s is the specific gravity of the particles and e is the void ratio. A critical condition sometimes occurs in silts or fine sands. If the upward velocity of flow increases beyond the critical hydraulic gradient, a quick condition develops.

Quicksands, if subjected to deformation or disturbance, can undergo a spontaneous loss of strength. This loss of strength causes them to flow like viscous liquids. For quicksand to develop, the sand or silt concerned must be saturated and loosely packed. Then, on disturbance, the constituent grains become more closely packed, thereby increasing the pore water pressure, which reduces the forces acting between the grains. This brings about a reduction in strength. If the pore water can escape very rapidly, the loss in strength is momentary. Hence, a quick condition requires that pore water does not readily escape. This is fulfilled if the sand/silt has a low permeability and/or the seepage path is long. Casagrande (1936) demonstrated that a critical porosity existed, above which a quick condition could be developed. He maintained that many coarse-grained sands, even when loosely packed, have porosities approximately equal to the critical condition,whereas medium- and fine-grained sands, especially if uniformly graded, exist well above the critical porosity when loosely packed. Accordingly, fine sands tend to be potentially more unstable than coarse-grained varieties, and finer sands also have lower permeabilities.

Quick conditions brought about by seepage forces are frequently encountered in excavations made in fine sands that are below the water table, for example, as in cofferdam work. As the velocity of the upward seepage force increases as the critical gradient is exceeded, the soil begins to boil more and more violently. Structures then fail by sinking into the quicksand. Liquefaction of potential quicksand may be caused by sudden shocks such as the action of heavy machinery (notably pile driving), blasting and earthquakes. Such shocks increase the stress carried by the pore water, the neutral stress, and give rise to a decrease in the effective stress and shear strength of the soil. A quick condition can also develop in a layered soil sequence where the individual beds have different permeabilities.

Piping refers to erosive action where sediments are removed by seepage forces, hence forming subsurface cavities and tunnels. For erosion tunnels to form, the soil must have some cohesion; the greater the cohesion, the wider the tunnel. In fact, fine sands and silts, especially those with dispersive characteristics, are most susceptible to piping failures. Obviously, the danger of piping occurs when the hydraulic gradient is high, that is, when there is a rapid loss of head over a short distance. For example, it has been associated with earth dams. As the pipe develops by backward erosion through a dam, it nears the source of water supply (i.e. the reservoir) so that eventually the water breaks into and rushes through the pipe (Fig. 5.8). Ultimately, the hole so produced collapses from lack of support.

Hydraulic uplift phenomena are associated with artesian conditions, that is, when water flowing under pressure through the ground is confined between two impermeable horizons (see the previous text). This can cause serious trouble in excavations, and both the position of the water table and the piezometric pressures should be determined before work commences. Otherwise, excavations that extend close to strata under artesian pressure may heave or, worse, may be severely damaged due to blow-outs taking place in the floors. Slopes also may fail. Indeed, such sites may have to be abandoned.

Groundwater Exploration

Groundwater investigation requires a thorough appreciation of the hydrology and geology of the area concerned, and a groundwater inventry needs to determine possible gains and losses affecting the subsurface reservoir. Of particular interest is the information concerning the lithology, stratigraphical sequence and geological structure, as well as the hydrogeological characteristics of the subsurface materials. Also of importance are the positions of the water table and piezometric level, and their fluctuations.

In major groundwater investigations, records of precipitation, temperatures, wind movement, evaporation and humidity may provide essential or useful supplementary information. Similarly, data relating to steamflow may be of value in helping to solve the groundwater equation since seepage into or from streams constitutes a major factor in the discharge or recharge of groundwater. The chemical and bacterial qualities of groundwater obviously require investigation.

Essentially, an assessment of groundwater resources involves the location of potential aquifers within economic drilling depths. Whether or not an aquifer will be able to supply the required amount of water depends on its thickness and spatial distribution, its porosity and permeability, whether it is fully or partially saturated and whether or not the quality of the water is acceptable. Another factor that has to be considered is pumping lift and the effect of drawdown on it.

The desk study involves a consideration of the hydrological, geological, hydrogeological and geophysical data available concerning the area in question. Particular attention should be given to assessing the lateral and vertical extent of any potential aquifers, to their continuity and structure, to any possible variations in formation characteristics and to possible areas of recharge and discharge. Additional information, relating to groundwater chemistry, the outflow of springs, surface run-off and data from pumping tests, from mine workings, from waterworks or meteorological data, should be considered. Information on vegetative cover, land utilization, topography and drainage pattern can prove of value at times.

Aerial photographs may aid recognition of broad rock and soil types and, thereby, help locate potential aquifers. The combination of topographical and geological data may help identify areas of likely groundwater recharge and discharge. In particular, the nature and extent of superficial deposits may provide some indication of the distribution of areas of recharge and discharge. Aerial photographs allow the occurrence of springs to be recorded.

Variations in water content in soils and rocks that may not be readily apparent on black and white photographs often are depicted by false colour. In fact, the specific heat of water is usually two to ten times greater than that of most rocks, and this, therefore, facilitates its detection in the ground. Indeed, the specific heat of water can cause an aquifer to act as a heat sink that, in turn, influences near-surface temperatures.

Also, the vegetative cover may be identifiable from aerial photographs and, as such, may provide some clue as to the occurrence of groundwater. In arid and semi-arid regions, in particular, the presence of phreatophytes, that is, plants that have a high transpiration capacity and derive water directly from the water table, indicates that the water table is near the surface. By contrast, xerophytes can exist at low moisture contents in soil, and their presence would suggest that the water table is at an appreciable depth. As a consequence, groundwater prediction maps sometimes can be made from aerial photographs. These can be used to help locate the sites of test wells.

Geological mapping frequently forms the initial phase of exploration and should identify potential aquifers such as sandstones and limestones, and distinguish them from aquicludes. Superficial deposits may perform a confining function in relation to any major aquifers they overlie, or because of their lithology, they may play an important role in controlling recharge to major aquifers. Furthermore, geological mapping should locate igneous intrusions and major faults. Obviously, it is important during the mapping programme to establish the geological structure.

Direct subsurface exploration techniques are dealt with in Chapter 7.

Geophysical Methods and Groundwater Investigation

As far as seismic methods are concerned, velocities in crystalline rocks are generally high to very high (Table 7.4). Velocities in sedimentary rocks increase concomitantly with consolidation and with increase in the degree of cementation and diagenesis. Unconsolidated sedimentary accumulations have maximum velocities varying as a function of the mineralogy, the volume of voids (either air-filled or water-filled) and grain size.

Porosity tends to lower the velocity of shock waves through a material. Indeed, the compressional wave velocity, V_p, is related to the porosity, n, of a normally consolidated sediment as follows:

$$\frac{1}{V_p} = \frac{n}{V_{pf}} + \frac{1-n}{V_{pl}} \tag{4.32}$$

where V_{pf} is the velocity in the pore fluid and V_{pl} is the compressional wave velocity for the intact material as determined in the laboratory. The compressional wave velocities may be raised appreciably by the presence of water.

The resistivity method does not provide satisfactory quantitative results if the potential aquifers being surveyed are thin, that is, 6 m or less in thickness, especially if they are separated by thick argillaceous horizons. In such situations, either cumulative effects are obtained or anomalous resistivities are measured, the interpretation of which is extremely difficult, if not impossible. In addition, the resistivity method is more successful when used to investigate a formation that is thicker than the one above it.

Most rocks and soils conduct electric current only because they contain water. But the widely differing resistivity of the various types of pore water can cause variations in the resistivity of soil and rock formations, ranging from a few tenths of an ohm-metre to hundreds of ohm-metres.

Moreover, the resistivity of water changes markedly with temperature, and temperature increases with depth. Hence, for each bed under investigation, the temperature of both rock and water must be determined or closely estimated, and the calculated resistivity of the pore water at that temperature should be converted to its value at a standard temperature (i.e. 25°C).

As the amount of water present in a rock is influenced by the porosity, the resistivity provides a measure of its porosity. For example, in granular materials in which there are no clay minerals, the relationship between the resistivity, ρ, on the one hand and the density of the pore water, ρ_w, the porosity, n, and the degree of saturation, S_r, on the other, is as follows:

$$\rho = a\rho_w n^{-x} S_r^{-y} \tag{4.33}$$

where a, x and y are variables (x ranges from 1.0 for sand to 2.5 for sandstone and y is approximately 2.0 when the degree of saturation is greater than 30%). If clay minerals do occur in sands or sandstones, then the resistivity of the pore water is reduced by ion exchange with the clay minerals so that this relationship becomes invalid. For those formations that occur below the water table and therefore are saturated, the above expression becomes:

$$\rho = a\rho_w n^{-x} \tag{4.34}$$

since $S_r = 1$ (i.e. 100%).

Generally speaking, magnetic and gravity methods are not used in groundwater exploration, except to derive a regional picture of the subsurface geology. They are referred to in Chapter 7, as is the geophysical logging of drillholes.

Maps

Isopachyte maps can be drawn to show the thickness of a particular aquifer and the depth below the surface of a particular bed. They can be used to estimate the positions and depths of drillholes. They also provide an indication of the distribution of potential aquifers.

Maps showing groundwater contours are compiled when there are a sufficient number of observation wells to determine the configuration of the water table (Fig. 4.3). Data on surface water levels in reservoirs and streams that have free connection with the water table also should be used in the production of such maps. These maps usually are compiled for the periods of the maximum, minimum and mean annual positions of the water table. A water table contour map is most useful for studies of unconfined groundwater.

As groundwater moves from areas of higher potential towards areas of lower potential and as the contours on groundwater contour maps represent lines of equal potential, the direction of groundwater flow moves from highs to lows at right angles to the contours. Analysis of conditions revealed by groundwater contours is made in accordance with Darcy's law. Accordingly, spacing of contours is dependent on the flow rate, and on aquifer thickness and permeability. If continuity of flow rate is assumed, then the spacing depends on aquifer thickness and permeability. Hence, areal changes in contour spacing may be indicative of changes in aquifer conditions. However, because of the heterogeneity of most aquifers, changes in gradient must be carefully interpreted in relation to all factors. The shape of the contours portraying the position of the water table helps to indicate where areas of recharge and discharge of groundwater occur. Groundwater mounds can result from the downward seepage of surface water. In an ideal situation, the gradient from the centre of such a recharge area decreases radially and at a declining rate. An impermeable boundary or change in transmissivity will affect this pattern.

Depth to water table maps show the depth to water from the ground surface. They are prepared by overlaying a water table contour map on a topographical map of the same area and scale, and recording the values at the points where the two types of contours intersect. Depth to water contours are then interpolated in relation to these points. A map indicating the depth to the water table also can provide an indication of areas of recharge and discharge. Both are most likely to occur where the water table approaches the surface.

Water-level-change maps are constructed by plotting the change in the position of the water table recorded at wells during a given interval of time. The effects of local recharge or discharge often appear as distinct anomalies on water-level-change maps. For example, a water-level-change map may indicate that the groundwater levels beneath a river have remained constant while falling everywhere else. This would suggest an influent relationship between the river and aquifer. Hence, such maps can help identify the locations where there are interconnections between surface water and groundwater. These maps also permit an estimation to be made of the change in groundwater storage that has occurred during the lapse in time involved.

Assessment of Field Permeability

An initial assessment of the magnitude and variability of the in situ permeability can be obtained from tests carried out in boreholes as the hole is advanced. By artificially raising the level of water in the borehole (falling head test) above that in the surrounding ground, the flow rate from the borehole can be measured. However, in very permeable soils, it may not be possible to raise the level of water in the borehole. Conversely, the water level in the borehole can be artificially depressed (rising head test), thereby allowing the rate of water flow into the borehole to be assessed. Wherever possible, rising and falling head tests should be carried out at each required level, and the results averaged.

In rising or falling head tests in which the piezometric head varies with time, the permeability is determined from the expression:

$$k = \frac{A}{F\left(t_2 - t_1\right)} \ln\left(\frac{h_1}{h_2}\right) \tag{4.35}$$

where h_1 and h_2 are the piezometric heads at times t_1 and t_2, respectively, A is the inner cross-sectional area of the casing in the borehole and F is an intake or shape factor. Where a borehole of diameter D is open at the base and cased throughout its depth, $F = 2.75\ D$. If the casing extends throughout the permeable bed to an impermeable contact, then $F = 2D$.

The constant head method of in situ permeability testing is used when the rise or fall in the level of the water is too rapid for accurate recording (i.e. occurs in less than 5 min). This test normally is conducted as an inflow test in which the flow of water into the ground is kept under a sensibly constant head (e.g. by adjusting the rate of flow into the borehole so as to maintain the water level at a mark on the inside of the casing near the top). The method only is applicable to permeable ground such as gravels, sands and broken rock, when there is a negligible or zero time for equalization. The rate of flow, Q, is measured once a steady flow into and out of the borehole has been attained over a period of some 10 min. The permeability, k, is derived from the following expression:

$$k = Q/Fh_c \tag{4.36}$$

where F is the intake factor and h_c is the applied constant head.

The permeability of an individual bed of rock can be determined by a water injection or packer test carried out in a drillhole. This is done by sealing off a length of uncased hole with packers and injecting water under pressure into the test section (Fig. 4.9). Usually, because it is more convenient, packer tests are carried out after the entire length of a hole has been drilled. Two packers are used to seal off selected test lengths, and the tests are performed from the base of the hole upwards. The hole must be flushed to remove sediment prior to a test being performed. With double packer testing, the variation in permeability throughout the test hole is determined. The rate of flow of water over the test length is measured under a range of constant pressures and recorded. The permeability is calculated from a flow–pressure curve. Water generally is pumped into the test section at steady pressures for periods of

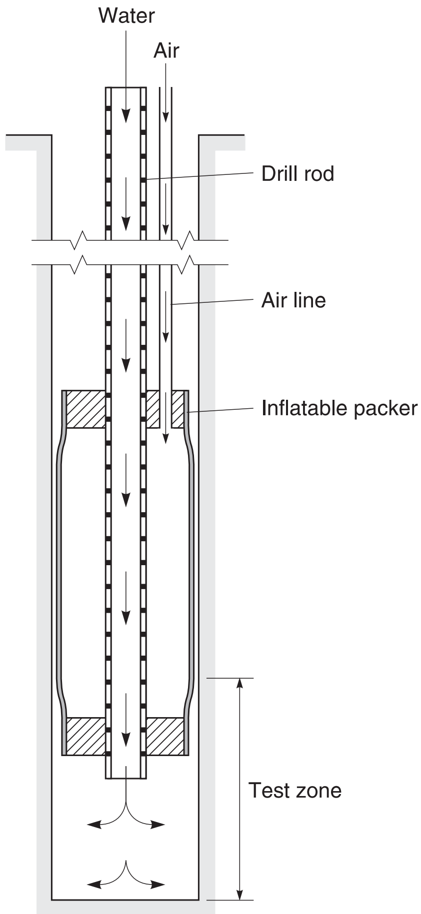


Figure 4.9

Drillhole packer test equipment. In the double packer test, the zone of rock to be tested in the drillhole is isolated by using two packers that seal off the drillhole, the water being pumped into the space between the packers. An alternative method that can be carried out as drilling proceeds is to use a single packer for testing the zone between the bottom of the packer and the base of the drillhole. The average flow under equilibrium conditions is obtained from a metered water supply, acting under a known pressure and gravity head.

15 min, readings of water absorption being taken every 15 min. The test usually consists of five cycles at successive pressures of 6, 12, 18, 12 and 6 kPa for every metre depth of packer below the surface. The evaluation of the "permeability" from packer tests is normally based upon methods that use a relationship of the form:

$$k = \frac{Q}{C_s r h}, \tag{4.37}$$

where Q is the steady flow rate under an effective applied head, h (corrected for friction losses), r is the radius of the drillhole and C_s is a constant depending upon the length and diameter of the test section.

Field pumping tests allow the determination of the coefficients of permeability and storage as well as the transmissivity of a larger mass of ground than the aforementioned tests. A pumping test involves abstracting water from a well at known discharge rates and observing the resulting water levels as drawdown occurs. At the same time, the behaviour of the water table in the aquifer can be recorded in observation wells radially arranged about the abstraction well. There are two types of pumping tests, namely, the constant-pumping-rate aquifer test and the step performance test. In the former test, the rate of discharge is constant, whereas in a step performance test, there are a number of stages, each of equal length of time, but at different rates of discharge. The step performance test usually is carried out before the constant-pumping-rate aquifer test. Yield drawdown graphs are plotted from the information obtained (Fig. 4.10). The hydraulic efficiency of the well is indicated by the nature of the curves, the more vertical and straighter they are, the more efficient the well.

Assessment of Flow in the Field

Flowmeters

A flowmeter log provides a record of the direction and velocity of groundwater movement in a drillhole. Flowmeter logging requires the use of a velocity-sensitive instrument, a system for lowering the instrument into the hole, a depth-measuring device to determine the position of the flowmeter and a recorder located at the surface.

The direction of flow of water is determined by slowly lowering and raising the flowmeter through a section of hole 6 to 9 m in length, and recording velocity measurements during both traverses. If the velocity measured is greater during the downward than the upward traverse, then the direction of flow is upward and vice versa.

A flowmeter log, made while a drillhole is being pumped at a moderate rate or by allowing water to flow if there is sufficient artesian head, permits identification of the zones contributing to the discharge. It also provides information on the thickness of these zones and the relative yield at that discharge rate. Because the yield varies approximately directly with the drawdown of water level in a well, flowmeter logs made by pumping should be pumped at least at three different rates. The drawdown of water level should be recorded for each rate.

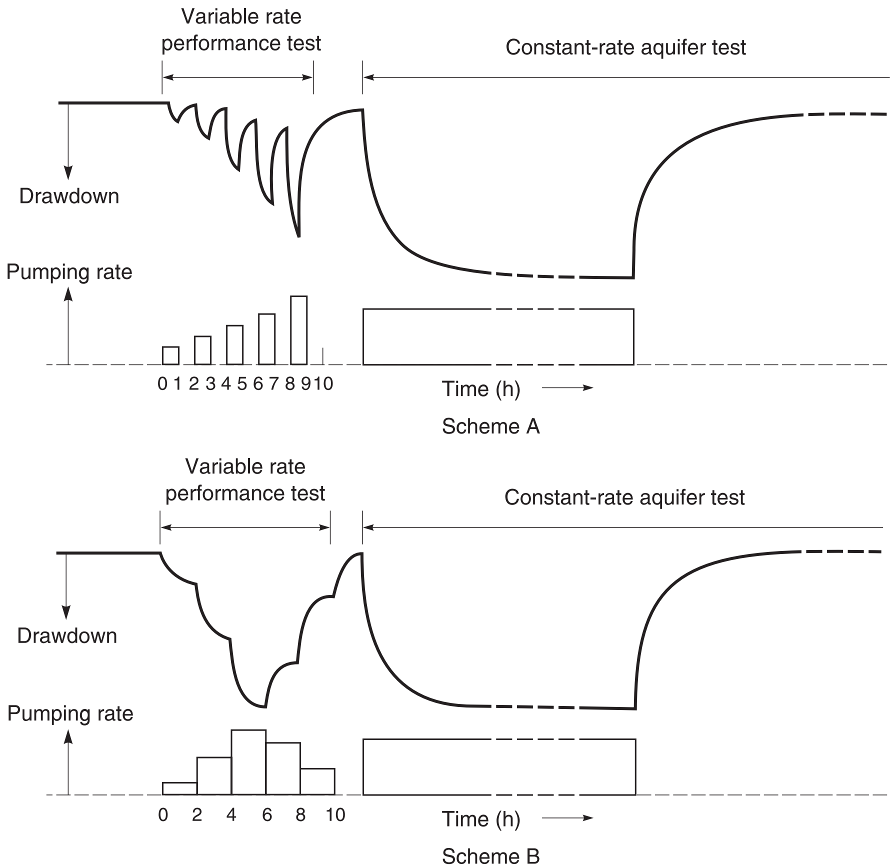


Figure 4.10

Yield drawdown curves from pumping tests.

Tracers

A number of different types of tracer have been used to investigate the movement of groundwater and the interconnection between surface and groundwater resources. The ideal tracer should be easy to detect quantitatively in minute concentrations. It should not change the hydraulic characteristics of, or be adsorbed by, the media through which it is flowing; it should be more or less absent from, and should not react with, the groundwater concerned and it should have low toxicity. The type of tracers in use include water-soluble dyes that can be detected by colorimetry, sodium chloride or sulphate salts that can be detected chemically and strong electrolytes that can be detected by electrical conductivity. Radioactive tracers also are used, and one of their advantages is that they can be detected in minute quantities in water. Such tracers should have a useful half-life and should present a minimum of hazard. For example, tritium is not the best of tracers because of its relatively long half-life. In addition, because it is introduced as tritiated water, it is adsorbed preferentially by any montmorillonite present.

When a tracer is injected via a drillhole into groundwater, it is subject to diffusion, dispersion, dilution and adsorption. Dispersion is the result of very small variations in the velocity of laminar flow through porous media. Molecular diffusion is negligible, unless the velocity of flow is unusually low. Even if these processes are not significant, flow through an aquifer may be stratified or concentrated along discontinuities. Therefore, a tracer may remain undetected unless the observation drillholes intersect these discontinuities.

The vertical velocity of water movement in a drillhole can be assessed by using tracers. A tracer is injected at the required depth, and the direction and rate of movement is monitored by a probe.

Determination of the permeability in the field can be done by measuring the time it takes for a tracer to move between two test holes. As with pumping tests, this tracer technique is based on the assumption that the aquifer is homogeneous and that observations taken radially at the same distance from the well are comparable. This method of assessing permeability requires that injection and observation wells be close together (to avoid excessive travel time) and that the direction of flow be known so that observation holes are correctly sited.

Flow Nets

Flow nets provide a graphical representation of the flow of water through the ground and indicate the loss of head involved (Fig. 4.11). They also provide data relating to the changes in head velocity and effective pressure that occur in a foundation subjected to flowing groundwater conditions. For example, where the flow lines of a flow net move closer together, this indicates that the flow increases, although their principal function is to indicate the direction of flow. The equipotential lines indicate equal losses in head or energy as the water flows through the ground, so that the closer they are, the more rapid is the loss in head. Hence, a flow net can provide quantitative data related to the flow problem in question, for example, seepage pressures can be determined at individual points within the net.

By using a flow net, it is possible to estimate the amount of water flowing through the soil. If the total loss of head and the permeability of the soil are known, then the quantity of water involved can be calculated by using Darcy's law. However, it is not really as simple as that, because the area through which the water flows usually varies, as does the hydraulic gradient, since the flow paths vary in length. By using the total number of flow paths, f, the total number

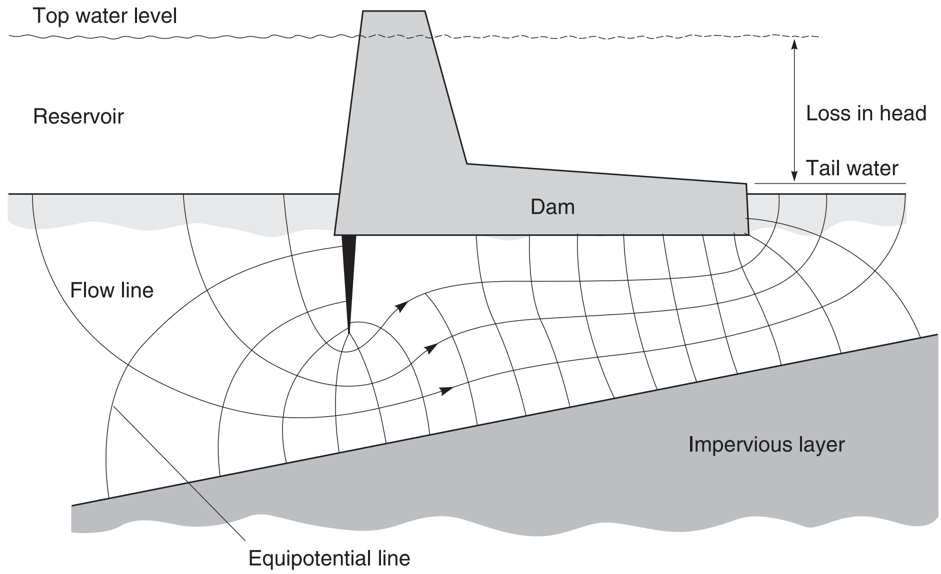


Figure 4.11

Flow net beneath a concrete gravity dam with cut-off at the heal, showing 17 equipotential drops and four flow channels.

of equipotential drops, *d*, and the total loss of head, i_t, together with the permeability, *k*, in the following expression:

$$Q = ki_t(f/d) \tag{4.38}$$

the quantity of water flow, *Q*, can be estimated.

Groundwater Quality

The quality must be considered in any assessment of groundwater resources (Anon, 1993). A study is required of changes in groundwater quality from outcrop areas to those where the aquifer is confined and also of changes that occur vertically within an aquifer. The chemical and biological characteristics of groundwater determine whether or not it can be used for domestic, industrial or agricultural supply. However, the number of major dissolved constituents in groundwater is quite limited, and the natural variations are not as great as might be expected. Nevertheless, groundwater is a complex chemical substance that owes its composition mainly to the solution of soluble constituents in, and chemical reactions between, the water and the rock or soil masses through which it travels (Table 4.6). Of critical importance in this context is the residence time of the groundwater, since this determines

Table 4.6. Chemical analysis of representative groundwaters

Location Aquifer	Gravesend Chalk	Watford Chalk	Bourne, Lincs. Great Oolite	Summerfield, Worcs. Sherwood Sandstone	Thornton, Northumberland Fell Sandstone
Ca	280	115	135	40	60
Mg	31	5	9	12	60
Na	2750	18			
K	98	15	4	8	
CO_3	153	147	138	56	
SO_4	700	39	150	26	38
Cl	5000	20	24	19	22
NO_3	35	ND	2	30	
TDS	9370	384	491	213	ND
CH	255	245	230	93	ND
N-CH	1755	64	145	97	ND

Note: TDS = total dissolved solids; CH = carbonate hardness; N-CH = non-carbonate hardness; ND not determined. Classification of TDS (mg l^{-1}): fresh = less than 1000; slightly saline = 1000 to 3000; moderately saline = 3000 to 10,000; very saline = 10,000 to 35,000; briny = over 35,000. Classification of hardness (mg l^{-1} as $CaCO_3$): soft = under 60; moderately hard = 60–120; hard = 120–180; very hard = over 180.

whether there is sufficient time for dissolution of minerals to proceed to the point where the solution is in equilibrium with the reaction. Residence time depends on the rate of groundwater movement, and this usually is very slow beneath the water table. As the character of groundwater in an aquifer frequently changes with depth, it is possible at times to recognize zones of different qualities of groundwater (Elliot et al., 2001).

The solution of carbonates, especially calcium and magnesium carbonate, is principally due to the formation of weak carbonic acid in the soil horizons where CO_2 is dissolved by soil water. Calcium in sedimentary rocks is derived from calcite, aragonite, dolomite, anhydrite and gypsum. In igneous and metamorphic rocks, calcium is supplied by the feldspars, pyroxenes, amphiboles and the less common minerals such as apatite and wollastonite. Because of its abundance, calcium is one of the most common ions in groundwater. Bicarbonate is formed when calcium carbonate is attacked by carbonic acid. Calcium carbonate and bicarbonate are the dominant constituents found in the zone of active circulation and for some distance under the cover of younger strata. The normal concentration of calcium in groundwater ranges from 10 to 100 mg l^{-1}. Such concentrations have no effect on health, and it has been suggested that as much as 1000 mg l^{-1}may be harmless (Edmunds and Smedley, 1996).

Magnesium, sodium and potassium are less common cations, and sulphate, chloride and nitrate (to some extent) are less common anions, although the latter may be present in significant concentrations in some groundwaters. Dolomite is the common source of magnesium in sedimentary rocks. The rarer evaporite minerals such as epsomite, kierserite, kainite and carnallite are not significant contributors. Olivine, biotite, hornblende and augite are among those minerals that make significant contributions in the igneous rocks, and serpentine, talc, diopside and tremolite are among the metamorphic contributors. Despite the higher solubilities of most of its compounds (magnesium sulphate and magnesium chloride are both very soluble), magnesium usually occurs in lesser concentrations in groundwaters than calcium. Common concentrations of magnesium range from about 1 to 40 mg l^{-1}, concentrations above 100 mg l^{-1} are rarely encountered.

Sodium does not occur as an essential constituent of many of the principal rock-forming minerals, plagioclase feldspar being the exception. Consequently, plagioclase is the primary source of most sodium in groundwater. Obviously, in areas of evaporitic deposits, halite is important. Sodium salts are highly soluble and do not precipitate unless concentrations of several thousand parts per million are reached. The only common mechanism for removal of large amounts of sodium ions from groundwater is through ion exchange that operates if the sodium ions are in great abundance. The conversion of calcium bicarbonate to sodium bicarbonate, no doubt, accounts for the removal of some sodium ions from sea water that has invaded freshwater aquifers. This process is reversible. All groundwaters contain measurable amounts of sodium, up to 20 mg l^{-1} being the most common concentrations.

Common sources of potassium are the feldspars and micas of the igneous and metamorphic rocks. Potash minerals such as sylvite occur in some evaporitic sequences, but their contribution is not important. Although the abundance of potassium in the Earth's crust is similar to that of sodium, its concentration in groundwater is usually less than a tenth that of sodium. Most groundwater contains less than 10 mg l^{-1}. As with sodium, potassium is highly soluble and therefore is not easily removed from groundwater except by ion exchange.

Sedimentary rocks such as shales and clays may contain pyrite or marcasite, from which sulphur can be derived. However, most sulphate ions probably are derived from the solution of calcium and magnesium sulphate minerals found in evaporitic sequences, gypsum and anhydrite being the most common. The concentration of the sulphate ion in groundwater can be affected by sulphate-reducing bacteria. The products of sulphate reduction are hydrogen sulphide and carbon dioxide. Hence, a decline in the sulphate ion frequently is associated with an increase in the bicarbonate ion. Concentration of sulphate in groundwater is usually less than 100 mg l^{-1} and may be less than 1 mg l^{-1} if sulphate-reducing bacteria are active.

The chloride content of groundwater may be due to the presence of soluble chlorides from rocks, saline intrusion, connate and juvenile waters or contamination by industrial effluent or domestic sewage. In the zone of circulation, the chloride ion concentration normally is relatively small. Chloride is a minor constituent in the Earth's crust; sodalite and apatite are the only igneous and metamorphic minerals containing chlorite as an essential constituent. Halite is one of the principal mineral sources. As in the case of sulphate, the atmosphere probably makes a significant contribution to the chloride content of surface waters. These, in turn, contribute to the groundwater. Usually, the concentration of chloride in groundwater is less than 30 mg l^{-1}, but concentrations of 1000 mg l^{-1} or more are common in arid regions.

Nitrate ions are generally derived from the oxidation of organic matter with a high protein content. Their presence may be indicative of a source of pollution, and their occurrence usually is associated with shallow groundwater sources. Concentrations in fresh water generally do not exceed 5 mg l^{-1}, although in rural areas where nitrate fertilizer is applied liberally, concentrations may exceed 600 mg l^{-1}.

Although silicon is the second most abundant element in the Earth's crust and is present in most of all the principal rock-forming minerals, its low solubility means that it is not one of the most abundant constituents of groundwater. It generally contains between 5 and 40 mg l^{-1}, although higher values may be recorded in groundwater from volcanic rocks.

Iron forms approximately 5% of the Earth's crust and is contained in a great many minerals in rocks, as well as occurring as ore bodies. Most iron in solution is ionized.

Ion exchange affects the chemical nature of groundwater. The most common natural cation exchangers are clay minerals, humic acids and zeolites. The replacement of Ca and Mg by Na may occur when groundwater moves beneath argillaceous rocks into a zone of more restricted circulation. This produces soft water. Changes in temperature–pressure conditions may result in precipitation, for instance, a decrease in pressure may liberate CO_2, causing the precipitation of calcium carbonate.

Certain dissolved gases such as oxygen and carbon dioxide alter groundwater chemistry; others affect the use of water. For example, hydrogen sulphide in concentrations of more than 1 mg l^{-1} renders groundwater unfit for human consumption because of the objectionable odour.

Wells

The commonest way of recovering groundwater is to sink a well and lift water from it (Fig. 4.12). The most efficient well is developed so as to yield the greatest quantity of groundwater with the least drawdown and the lowest velocity in the vicinity of the well. The specific capacity of a well is expressed in litres of yield per metre of drawdown when the well is being pumped. It is indicative of the relative permeability of the aquifer. Location of a well obviously is important

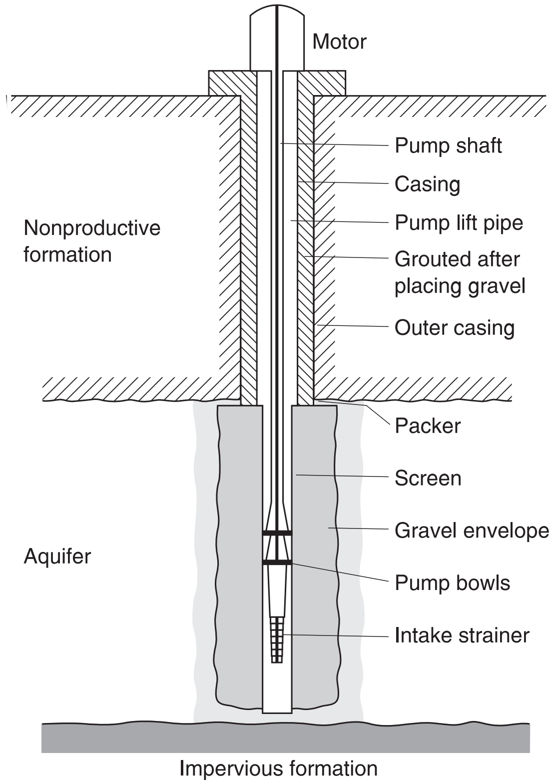


Figure 4.12

Gravel-packed well installation.

if an optimum supply is to be obtained. A well site should be selected after a careful study of the geological setting.

Completion of a well in an unconsolidated formation requires that it be cased, so that the surrounding deposits are supported. Sections of the casing must be perforated to allow the penetration of groundwater from the aquifer into the well, or screens can be used. The casing should be as permeable or more permeable than the deposits it confines.

Wells that supply drinking water should be properly sealed. However, an important advantage of groundwater is its normal comparative freedom from bacterial pollution. For example, groundwater that has percolated through fine-grained sands usually is cleared of bacterial pollution within about 30 m of the flow path. On the other hand, flow through open jointed limestone may transmit pollution for considerable distances. Abandoned wells should be sealed to prevent aquifers being contaminated.

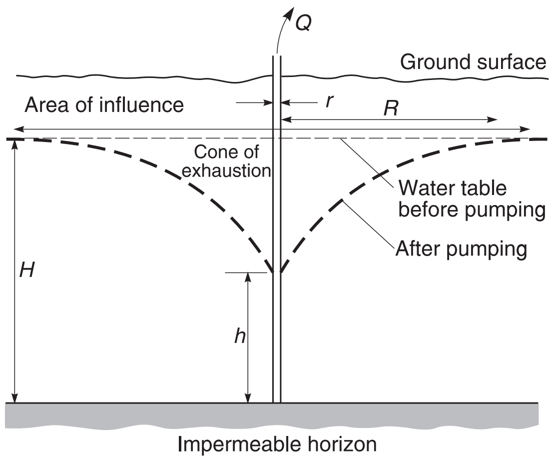


Figure 4.13

Cone of depression or exhaustion developed around a pumped well in an unconfined aquifer. $Q=\pi k(H^2-h^2)/(\ln R/r)$. Q=quantity; k=coefficient of permeability.

The yield from a well in granular material can be increased by surging, which removes the finer particles from the zone about the well. Water supply from wells in rock can be increased by driving galleries or adits from the bottom of deep wells. Yields from rock formations also can be increased by fracturing the rocks with explosives or with fluid pumped into the well under high pressure or, in the case of carbonate rocks such as chalk, by using acid to enlarge the discontinuities. The use of explosives in sandstones has led to increases in yield of up to 40%, whereas acidification of wells in carbonate rocks has increased yields by over 100%.

When water is abstracted from a well, the water table in the immediate vicinity is lowered and assumes the shape of an inverted cone, which is named the cone of depression (Fig. 4.13). The steepness of the cone is governed by the soil or rock type, it being flatter in highly permeable materials such as openwork gravels than in the less permeable chalk. The size of the cone of depression depends on the rate of pumping, equilibrium being achieved when the rate of abstraction balances the rate of recharge. But if abstraction exceeds recharge, then the cone of depression increases in size, and its gradual extension from the well may mean that shallow wells within its area of influence dry up. If these shallow wells are to continue to be of use, then the offending well should be rested so that the water table may regain its former level. Otherwise, they must be sunk deeper, which only accentuates the problem further, since this means a further depression of the water table.

The development of wells for groundwater supplies in rural areas, especially in developing countries, frequently is of major importance. In such areas, fractured zones and weathered horizons of granitic or gneissic masses may provide sufficient water for small communities.

In addition, the fractured and weathered contact zones of thick dykes and sills may yield similar quantities. For example, Bell and Maud (2000) refer to the four categories of well yield recognized by the South African Department of Water Affairs. These are high well yields (over 3.0 l s^{-1}) that are suitable for the supply of medium- to large-scale water schemes supporting small towns and/or small- to medium-scale irrigation schemes. Moderate well yields (0.5 to 3.0 l s^{-1}) are suitable for reticulation schemes for villages, clinics and schools. Low well yields (0.1 to 0.5 l s^{-1}) can be used to supply a hand pump for a non-reticulating water supply for a small community and stock watering purposes. Lastly, very low well yields (less than 0.1 l s^{-1}) only provide marginal supplies.

Safe Yield

The abstraction of water from the ground at a greater rate than it is being recharged leads to a lowering of the water table and upsets the equilibrium between discharge and recharge. The concept of safe yield has been used to express the quantity of water that can be withdrawn from the ground without impairing an aquifer as a water source. Draft in excess of safe yield is overdraft. This can give rise to pollution or cause serious problems due to severely increased pumping lift. Indeed, this eventually may lead to the exhaustion of a well.

Estimation of the safe yield is a complex problem that must take into account the climatic, geological and hydrological conditions. As such, the safe yield is likely to vary appreciably with time. Nonetheless, the recharge–discharge equation, the transmissivity of the aquifer, the potential sources of pollution and the number of wells in operation must all be given consideration if an answer is to be found. The safe yield, G, often is expressed as follows:

$$G = P - Q_s - E_T + Q_g - \Delta S_g - \Delta S_s \tag{4.39}$$

where P is the precipitation on the area supplying the aquifer, Q_s is the surface stream flow over the same area, E_T is the evapotranspiration, Q_g is the net groundwater inflow to the area, ΔS_g is the change in groundwater storage and ΔS_s is the change of surface storage. With the exception of precipitation, all the terms of this expression can be subjected to artificial change. The equation cannot be considered an equilibrium equation or solved in terms of mean annual values. It can be solved correctly only on the basis of specified assumptions for a stated period of years.

Transmissivity of an aquifer may place a limit on the safe yield, even though this equation may indicate a potentially large draft. This can only be realised if the aquifer is capable of transmitting groundwater from the source area to the wells at a rate high enough to sustain the draft. Where the pollution of groundwater is possible, then the location of wells, their type and the rate of abstraction must be planned in such a way that conditions permitting pollution cannot be developed.

Once an aquifer is developed as a source of water supply, then effective management becomes increasingly necessary if it is not to suffer deterioration. Moreover, management should not merely be concerned with the abstraction of groundwater but also should consider its utilization, since different qualities of water can be put to different uses. Pollution of water supply is most likely to occur when the level of the water table has been so lowered that all the water that goes underground within a catchment area drains quickly and directly to the wells. Such lowering of the water table may cause reversals in drainage, so that water drains from rivers into the groundwater system, rather than the other way around. This river water may be polluted.

Artificial Recharge

Artificial recharge may be defined as an augmentation of the natural replenishment of groundwater storage by artificial means. Its main purpose is water conservation, often with improved quality as a second aim. For example, soft river water may be used to reduce the hardness of groundwater. Artificial recharge therefore is used for reducing overdraft, for conserving and improving surface run-off and for increasing available groundwater supplies.

The suitability of a particular aquifer for artificial recharge must be investigated. For example, it must have adequate storage, and the bulk of the water recharged should not be lost rapidly by discharge into a nearby river. The hydrogeological and groundwater conditions must be amenable to artificial replenishment. An adequate and suitable source of water for recharge must be available. The source of water for artificial recharge may be storm run-off, river or lake water, water used for cooling purposes, industrial waste water or sewage water. Many of these sources require some kind of pre-treatment.

Interaction between artificial recharge and groundwater may lead to precipitation, for example, of calcium carbonate and iron and manganese salts, resulting in a lower permeability. Nitrification or denitrification, and possibly even sulphate reduction, may occur during the early stages of infiltration. Bacterial action may lead to the development of sludges that reduce the rate of infiltration.

There are several advantages of storing water underground. Firstly, the cost of artificial recharge may be less than the cost of surface reservoirs, and water stored in the ground is not subjected to evapotranspiration. Secondly, the likelihood of pollution is reduced. Thirdly, an aquifer will sometimes act as a distribution system, recharge water moving from one area to another as groundwater at depth. Fourthly, underground storage is important where suitable sites are not available at the surface. Lastly, temperature fluctuations of water stored underground are reduced.

Artificial recharge may be accomplished by various surface spreading methods utilizing basins, ditches or flooded areas; by spray irrigation or by pumping water into the ground via vertical shafts, horizontal collector wells, pits or trenches. The most widely practised methods are those of water spreading that allow increased infiltration to occur over a wide area when the aquifer outcrops at or near the surface. Therefore, these methods require that the ground has a high infiltration capacity. In the basin method, water is contained in a series of basins formed by a network of dykes, constructed to take maximum advantage of local topography.

Groundwater Pollution

Pollution can be defined as an impairment of water quality by chemicals, heat or bacteria to a degree that does not necessarily create an actual public health hazard, but does adversely affect waters for domestic, farm, municipal, commercial or industrial use. Contamination denotes impairment of water quality by chemical or bacterial pollution to a degree that creates an actual hazard to public health.

The greatest danger of groundwater pollution is from surface sources, including animal manure, sewage sludge, leaking sewers, polluted streams and refuse-disposal sites. Areas with a thin cover of superficial deposits or where an aquifer is exposed, such as a recharge area, are the most critical from the point of view of pollution potential. Any possible source of pollution or contamination in these areas should be carefully evaluated, both before and after any groundwater supply well is constructed and the viability of groundwater protection measures are considered (Hiscock et al., 1995). One approach to groundwater quality management is to indicate areas with high pollution potential on a map and to pay particular attention to activities within these vulnerable areas.

The attenuation of a pollutant as it enters and moves through the ground occurs as a result of biological, chemical and physical processes. Hence, the self-cleansing capacity of a soil or rock aquifer system depends on the physical and chemical attributes of the pollutant, the nature of the soil or rock comprising the aquifer and the way in which the pollutant enters the ground. In general, the concentration of a pollutant decreases as the distance it has travelled through the ground increases. However, it should be appreciated that the slow rate of travel of pollutants in underground strata means that a case of pollution may go undetected for a number of years.

The form of the pollutant is clearly an important factor with regard to its susceptibility to the various purifying processes. For instance, pollutants that are soluble, such as fertilizers and some industrial wastes, cannot be removed by filtration. Metal solutions may not be susceptible to biological action. Solids, on the other hand, are amenable to filtration, provided that the transmission media are not coarse-grained, fractured or cavernous. Karst or cavernous limestone areas pose particular problems in this respect. Insoluble liquids such as hydrocarbons are generally transmitted through porous media, although some fraction may be retained in the media. Usually, however, the most dangerous forms of groundwater pollution are those that are miscible with the water in the aquifer.

Concentrated sources of pollution are most undesirable because the self-cleansing ability of the ground in that area is likely to be exceeded. As a result, the “raw” pollutant may be able to enter an aquifer and travel some considerable distance from the source before being reduced to a negligible concentration. A much greater hazard exists when the pollutant is introduced into an aquifer beneath the soil horizon, since the powerful purifying processes that take place within the soil are bypassed and attenuation of the pollutant is reduced. This is most critical when the pollutant is added directly to the zone of saturation, because in most soils and rocks, the horizontal component of permeability usually is much greater than the vertical one. Consequently, a pollutant can then travel a much greater distance before significant attenuation occurs. This type of hazard often arises from poorly maintained domestic septic tanks and soakaways, from the discharge of quarry wastes, farm effluents and sewage into surface water courses and from the disposal of refuse and commercial wastes.

It generally is assumed that bacteria move at a maximum rate of about two-thirds the water velocity. Since most groundwater only moves at the rate of a few metres per year, the distances travelled by bacteria are usually quite small and, in general, it is unusual for bacteria to spread more than 33 m from the source of the pollution. However, Brown et al. (1972) suggested that viruses are capable of spreading over distances that exceed 250 m, although 20 to 30 m may be a more typical figure. Of course, in porous gravel, cavernous limestone or fissured rock, bacteria and viruses may spread over distances measured in kilometres.

Induced infiltration occurs where a stream is hydraulically connected to an aquifer and lies within the area of influence of a well (Fig. 4.14). When the well is overpumped, a cone of depression develops and spreads. Eventually, the aquifer may be recharged by the influent seepage of surface water, so that some proportion of the pumpage from the well then is obtained from the surface source. Induced infiltration is significant from the point of view of groundwater pollution in two respects. Firstly, hydraulic gradients may result in pollutants travelling in the opposite direction from that normally expected. Secondly, surface water resources are often less pure than the underlying groundwater; hence the danger of pollution

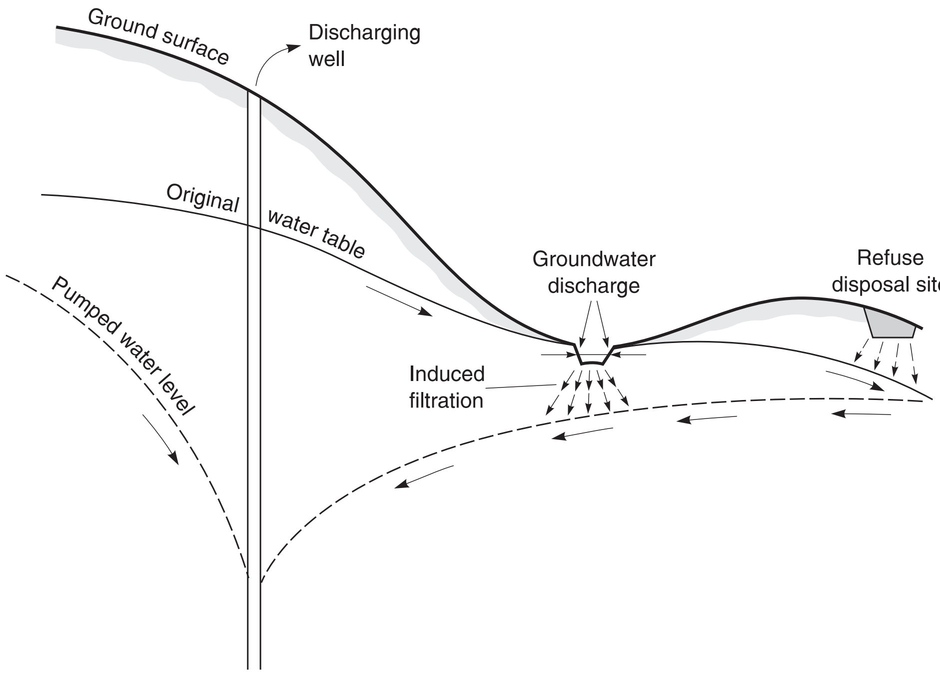


Figure 4.14

An example of induced infiltration brought about by overpumping. The original hydraulic gradient over much of the area has been reversed so that pollutants can travel in the opposite direction, namely, towards the well. Additionally, the aquifer has become influent (i.e. water drains from the river into the aquifer) instead of effluent as it was originally.

is introduced. However, induced infiltration does not automatically cause pollution, and it is a common method of augmenting groundwater supplies.

A list of potential groundwater pollutants would be almost endless, although one of the most common sources is sewage sludge (Andrews et al., 1998). This material arises from the separation and concentration of most of the waste materials found in sewage. Since sludge contains nitrogen and phosphorus, it has value as a fertilizer. Although this does not necessarily lead to groundwater pollution, the presence in sludge of contaminants such as heavy metals, nitrates, persistent organic compounds and pathogens does mean that the practice must be carefully controlled. The widespread use of chemical and organic pesticides or herbicides is another possible source of groundwater contamination (Chilton et al., 1998, 2005).

The disposal of wastes in landfill sites leads to the production of leachate and gases, which may present a health hazard as a consequence of pollution of groundwater supply. Leachate often contains high concentrations of dissolved organic substances resulting from the decomposition of organic material such as vegetable matter and paper. Site selection for waste disposal must take into account the character of the material that is likely to be tipped.

For instance, toxic or oily liquid waste represents a serious risk, although sites on impermeable substrata often merit a lower assessment of risk. Therefore, selection of a landfill site for a particular waste or a mixture of wastes involves a consideration of the geological and hydrogeological conditions. Argillaceous sedimentary, massive igneous and metamorphic rocks have low permeabilities, and therefore, afford the most protection to water supply (Bell et al., 1996). By contrast, the least protection is offered by rock masses intersected by open discontinuities or in which solution features are developed, or by open-work gravel deposits.

Leachate pollution can be tackled by either concentrating and containing, or by diluting and dispersing. Infiltration through sandy ground of liquids from a landfill may lead to their decontamination and dilution. Hence, sites for disposal of domestic refuse can be chosen where decontamination has the maximum chance of reaching completion and where groundwater sources are located far away enough to enable dilution to be effective. Consequently, domestic waste can be tipped at dry sites on sandy material that has a thickness of at least 15 m. Water supply sources should be located at least 0.8 km away from the landfill site. They should not be located on discontinuous rocks unless overlain by 15 m of clay deposits. Potential toxic waste should be contained. Such sites should be underlain and confined by at least 15 m of impermeable strata, and any source abstracting groundwater for domestic use should be at least 2 km away. Furthermore, the topography of the site should be such that run-off can be diverted from the landfill, so that it can be disposed of without causing pollution to surface waters. Containment can be achieved by an artificial impermeable lining placed over the base of a site. Drains can be installed beneath a landfill to convey leachate to a sump, which then can be either pumped to a sewer, transported away by tanker or treated on site.

Cemeteries form a possible health hazard. Decomposing bodies produce fluids that can leak to the water table if a leakproof coffin is not used. The leachate produced from a single grave is of the order of 0.4 $m^3\ a^{-1}$, and this may constitute a threat for about 10 years. The minimum distance required by law in England between a potable-water well and a cemetery is 91.4 m (100 yards). However, a distance of around 2500 m is better because the purifying processes in the soil can sometimes break down.

Run-off from roads can contain chemicals from many sources, including those that have been dropped, spilled or deliberately spread on the road. For instance, hydrocarbons from petroleum products and chlorides from de-icing agents are potential pollutants. There also is the possibility of accidents involving vehicles carrying large quantities of chemicals.

According to Mackay (1998), volatile organic chemicals (VOCs) are the most frequently detected organic contaminants in water supply wells in the United States. Of the VOCs, by far the most common are chlorinated hydrocarbon compounds. Conversely, petroleum hydrocarbons are rarely present in supply wells. This may be due to their in situ biodegradation.

Many of the VOCs are liquids and usually are referred to as non-aqueous phase liquids (NAPLs), which are sparingly soluble in water. Those that are lighter than water, such as the petroleum hydrocarbons, are termed LNAPLs, whereas those that are denser than water, such as the chlorinated solvents, are called DNAPLs. Of the VOCs, the DNAPLs are the least amenable to remediation (Acworth, 2001). Depending on the hydrogeological conditions, DNAPLs may percolate downwards into the saturated zone if they penetrate the ground in a large enough quantity. This can occur in coarse soils or discontinuous rock masses. Plumes of dissolved VOCs develop from the source of pollution. Although dissolved VOCs migrate at lesser speeds than the average velocity of groundwater, there are many examples of chlorinated VOC plumes several kilometres in length in the United States occurring in sand and gravel aquifers. Such plumes contain billions of litres of contaminated water. Because VOCs are sparingly soluble in water, the time taken for complete dissolution, especially of DNAPLs, by groundwater flow in coarse soils is estimated to be decades or even centuries.

Nitrate pollution is basically the result of intensive cultivation due to the large quantity of synthetic nitrogenous fertilizer used, although over-manuring with natural organic fertilizer can have the same result (Foster, 2000). Rapid transformation into nitrate results in an ion that, because it is neither adsorbed by nor precipitated in the soil, becomes easily leached by heavy rainfall and infiltrating water. However, the nitrate does not have an immediate effect on groundwater quality, possibly because most of the leachate that percolates through the unsaturated zone as intergranular seepage has a typical velocity of about 1 m per year. Thus, there may be a considerable delay between the application of the fertilizer and the subsequent increase in the concentration of nitrate in groundwater. This time lag, which is frequently of the order of 10 years or more, makes it very difficult to correlate fertilizer application with increased concentration of nitrate in groundwater. Hence, although nitrate levels may be unacceptably high now, they may worsen in the future because of the increasing use of nitrogenous fertilizers.

There are at least two ways in which nitrate pollution of water is known or suspected to be a threat to health. Firstly, the build-up of stable nitrate compounds in the bloodstream reduces its oxygen-carrying capacity. Infants under one year of age are at most risk. Consequently, a limit of 50 mg l^{-1} of nitrate (NO_3) has been recommended by the World Health Organisation (Anon, 1993). Secondly, the possible combination of nitrates and amines through the action of bacteria in the digestive tract results in the formation of potentially carcinogenic nitrosamines.

Measures that can be taken to alleviate nitrate pollution include better land-use management, mixing of water from various sources or the treatment of high-nitrate water before it is put into supply (Sigram et al., 2005). In general, the ion exchange process has been recommended as the preferred means of treating groundwater, although this may not be considered cost effective for all sources.

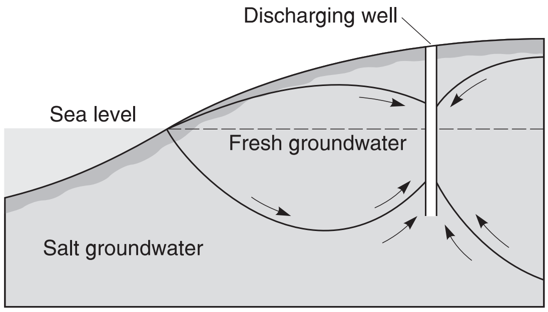


Figure 4.15

Saline intrusion occurring in pumped coastal aquifer.

Excessive lowering of the water table along a coast as a consequence of over abstraction can lead to saline intrusion. The salt water enters the aquifer via submarine outcrops, thereby displacing fresh water. However, the fresh water still overlies the saline water and continues to flow from the aquifer to the sea. The encroachment of salt water may extend for several kilometres inland, leading to the abandonment of wells. The first sign of saline intrusion is likely to be a progressively upward trend in the chloride concentration of water obtained from the affected wells. Typically, chloride levels may increase from a normal value of around 25 mg l^{-1} up to something approaching 19,000 mg l^{-1}, compared with a recommended upper limit for drinking water in Europe of 200 mg l^{-1} (Anon, 1980). Generally, saline water is drawn up towards the well and this sometimes is termed upcoming (Fig. 4.15). This is a dangerous condition that can occur even if the aquifer is not overpumped, and a significant proportion of the fresh water flow still reaches the sea. A well may be ruined by an increase in salt content even before the actual “cone” reaches the bottom of the well. This is due to leaching of the interface by fresh water. Once intrusion develops, it is not easy to control. The slow rates of groundwater flow, the density differences between fresh and salt waters and the flushing required usually mean that pollution, once established, may take many years to remove under natural conditions. Reduction of pumping to eliminate overdraft or artificial recharging have been used as methods of controlling saline intrusion. McDonald et al. (1998) described the use of resistivity tomography and ground conductivity surveys to delineate saline intrusion in a tidal coastal wetland in the south of England.

Irrigation water may pose a pollution hazard to groundwater, especially in arid and semi-arid regions where soluble salts may be present in soil. These salts can be leached from the soil and, hence, become concentrated in irrigation water, the situation being worsened if poor-quality water is used for irrigation purposes. Indeed, salinization of soils is a problem in many parts of the world. In such instances, shallow groundwater may be recharged by irrigation water. Hibbs and Boghici (1999) referred to two areas along the Rio Grande in Texas, where

Table 4.7. Composition of acid mine water from a South African coalfield

Determinant (mg l^{-1}, where appropriate)	Sample 1	Sample 2	Sample 3	Sample 4
TDS	4844	2968	3202	2490
Suspended solids	33	10.4	12	10.0
EC (mS m^{-1})	471	430	443	377
pH value	1.9	2.4	2.95	2.9
Turbidity as NTU	5.5	0.6	2.0	0.9
Nitrate NO_3 as N	0.1	0.1	0.1	0.1
Chlorides as Cl	310	431	406	324
Fluoride as F	0.6	0.5	0.33	0.6
Sulphate as SO_4	3250	1610	1730	1256
Total hardness	–	484	411	576
Calcium hardness as $CaCO_3$	–	285	310	327
Calcium Ca	173.8	114.0	124	131
Magnesium Mg	89.4	48.4	49.5	60.5
Sodium Na	247.0	326.0	311	278
Potassium K	7.3	9.4	8.9	6.4
Iron Fe	248.3	128	140	87
Manganese Mn	17.9	15	9.9	13.4
Aluminium Al	–	124	–	112

shallow aquifers have been adversely affected by the intensive use of irrigation water. They noted that there is a tendency for the salinity of the groundwater in the aquifer to increase downstream, the TDS increasing from between 1000 and 3500 mg l^{-1} to between 3000 and 6000 mg l^{-1}.

Acid mine drainage is produced when natural oxidation of sulphide minerals, notably pyrite, occurs in mine rock or waste that are exposed to air and water (Bullock and Bell, 1995). This is a consequence of the oxidation of sulphur in the mineral concerned to a higher oxidation state, with the formation of sulphuric acid and sulphate, and if aqueous iron is present and unstable, to the precipitation of ferric iron with iron hydroxide. Acid mine drainage, however, does not occur if the sulphide minerals are nonreactive or if the host rock contains sufficient alkaline material to neutralize the acidity. Acid generation can lead to elevated levels of heavy metals and sulphate in the affected water that obviously have a detrimental effect on its quality (Table 4.7).

Acid mine drainage from underground mines generally appears at the surface as point discharges (Bell et al., 2002). Acid mine drainage also can develop from surface sources such as mine waste. A major source of acid mine drainage may result from the closure of a mine.

Figure 4.16

Vegetation killed by acid mine drainage seeping from a shallow abandoned mine, Witbank Coalfield, South Africa.

When a mine is abandoned and dewatering by pumping ceases, the groundwater level rebounds. The workings, however, often act as drainage systems, so that the groundwater does not rise to its former level. Consequently, a residual dewatered zone remains that is subject to continuing oxidation. Groundwater may drain to the surface from old drainage adits, faults, springs and shafts that intercept rock in which groundwater is under artesian pressure. Hence, those streams receiving drainage from abandoned mines are often chronically polluted. This can have a notable impact on the aquatic environment, as well as vegetation (Fig. 4.16).

There are three key strategies in acid mine drainage management, namely, control of the acid generation process, control of acid migration, and collection and treatment of acid mine drainage. Obviously, the best solution is to control acid generation. Source control of acid mine drainage involves measures to prevent or inhibit oxidation, acid generation or contaminant leaching. If acid generation is prevented, then there is no risk of the contaminants entering the environment. Such control methods involve the removal or isolation of sulphide material, or the exclusion of water or air. The latter is much more practical and can be achieved by air-sealing adits in mines, or by placing a cover over acid-generating material, such as wastes.

Migration control is considered when acid generation is occurring and cannot be inhibited. Since water is the transport medium, control relies on the prevention of water entry to the source of acid mine drainage. Release control is based on measures to collect and treat either or both ground and surface acid mine drainage. In some cases, especially in working mines, this is the only practical option available. Treatment processes have concentrated on neutralization to raise the pH and precipitate metals. Lime or limestone commonly is used, although offering only a partial solution to the problem. Jarvis et al. (2003) described the use of settlement lagoons and a wetland to treat mine water.

As discussed, acid generation may occur in the surface layers of spoil heaps where air and water have access to sulphide minerals. Tailings deposits that have a high content of sulphide represent another potential source of acid generation. However, the low permeability of many tailings deposits, together with the fact that they commonly are flooded, means that the rate of acid generation and release is limited, but it can continue to take place long after a tailings deposit has been abandoned.

A variety of waste waters and process effluents arise during coal mining operations (Bell and Kerr, 1993). These may be produced by the actual extraction process, by the subsequent preparation of the coal or from the disposal of colliery spoil. The mineralogical character of the coal and spoil, and the washing processes employed, all affect the type of effluent produced. The major pollutants generally associated with coal mining are suspended solids, dissolved salts (particularly chlorides), acidity and iron compounds. A high level of mineralization is characteristic of many coal mining discharges and is reflected in the high values of electrical conductivity (values of 335,000 $\mu S\ cm^{-1}$ have been recorded). Not all minewaters, however, are highly mineralized. Elevated levels of suspended matter are associated with most coal mining effluents. The extraction of coal, particularly from opencast sites and from drift mines, may lead to the discharge of high loads of silt and fine coal particles into rivers.

The routine monitoring of groundwater level and water quality provides an early warning of pollution incidents. The first important step in designing an efficient groundwater-monitoring system is gaining a proper understanding of the mechanics and dynamics of pollutant propogation, the nature of the controlling flow mechanism and the aquifer characteristics. There should be a sufficient number of drillholes to allow the extent, configuration and concentration of a pollution plume to be determined. Furthermore, construction of a groundwater quality monitoring well must be related to the geology of the site, in particular, the well structure should not react with the groundwater if water quality is being monitored. These wells frequently are constructed using inert plastic casings and screens. Monitoring also can be carried out by using geophysical methods, especially resistivity surveys and ground-probing radar (McDowell et al., 2002). A methodology for delineating groundwater protection zones against pollution has been discussed by Bussard et al. (2006), which is based on the complete groundwater flow cycle. In this way, zones of groundwater source recharge can be defined, thereby allowing targeting of remediation programmes.

Description, Properties and Behaviour of Soils and Rocks

Soil Classification

Any system of soil classification involves grouping the different soil types into categories that possess similar properties and, in so doing, providing the engineer with a systematic method of soil description. Casagrande (1948) advanced one of the first comprehensive engineering classifications of soil. In the Casagrande system, the coarse-grained soils are distinguished from the fine on the basis of particle size. Gravels and sands are the two principal types of coarse-grained soils and, in this classification, both are subdivided into five subgroups on the basis of grading (Table 5.1). Well-graded soils are those in which the particle size distribution extends over a wide range without excess or deficiency in any particular sizes, whereas in uniformly graded soils, the distribution extends over a very limited range of particle sizes. In poorly graded soils, the distribution contains an excess of some particle sizes and a deficiency of others. A plasticity chart is used when classifying fine-grained soils, that is, silts and clays (see Table 5.2).

Table 5.1. Symbols used in the Casagrande soil classification

Main soil type		**Prefix**
Coarse-grained soils	Gravel	G
	Sand	S
Fine-grained soils	Silt	M
	Clay	C
	Organic silts and clays	O
Fibrous soils	Peat	Pt
Subdivisions		**Suffix**
For coarse-grainedsoils	Well graded, with little or no fines	W
	Well graded with suitable clay binder	C
	Uniformly graded with little or no fines	U
	Poorly graded with little or no fines	P
	Poorly graded with appreciable fines or well graded with excess fines	F
For fine-grained soils	Low compressibility (plasticity)	L
	Medium compressibility (plasticity)	I
	High compressibility (plasticity)	H

Table 5.2. Unified Soil Classification. Coarse soils. More than half of the material is larger than No. 200 sieve size†

Field identification procedures (excluding particles larger than 76 mm and basing fractions on estimated weights)			**Group symbols***	**Typical names**
Gravels. More than half of coarse fraction is larger than No. 7 sieve size*	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well-graded gravels, gravel–sand mixtures, little or no fines
		Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel–sand mixtures, little or no fines
	Gravels with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	GM	Silty gravels, poorly graded gravel–sand–silt mixtures
		Plastic fines (for identification procedures, see CL below)	GC	Clayey gravels, poorly graded gravel–sand–clay mixtures
Sands. More than half of coarse fraction is smaller than No. 7 sieve size‡	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well-graded sands, gravelly sands, little or no fines
		Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Poorly graded sands, gravelly sands,little or no fines
	Sands with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures, see ML below)	SM	Silty sands, poorly graded sand–silt mixtures
		Plastic fines (for identification procedures, see CL below)	SC	Clayey sands, poorly graded sand–clay mixtures

*Boundary classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example, GW–GC, well-graded gravel–sand mixture with clay binder;
† All sieve sizes on this chart are US standard. The No. 200 sieve size is about the smallest particle visible to the naked eye;
‡ For visual classification, the 6.3 mm size may be used as equivalent to the No. 7 sieve size.
Field identification procedure for fine-grained soils or fractions: These procedures are to be performed on the minus No. 40 sieve-size particles, approximately 0.4 mm. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.
Dilatancy (reacting to shaking): After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about 1 cm^3. Add enough water if necessary to make the soil soft but not sticky. Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat, which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens and, finally, it cracks and crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil. Very fine clean sands give the quickest and most distinct reaction, whereas a plastic clay has no reaction. Inorganic silts, such as typical rock flour, show a moderately quick reaction.
Dry strength (crushing characteristic): After removing particles larger than No. 40 sieve size, mould a pat of soil to the

Table 5.2.—Cont'd.

Information required for describing soils		Laboratory classification criteria		
Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information: and symbols in parenthesis For undistributed soils add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics Example: Silty sand, gravelly; about 20% hard, angular gravel particles 12.5 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)	Use grain size curve in identifying the fractions as given under field identification	Determine percentages of gravel and sand from grain-size curve. Depending on fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows: Less than 5%: GW, GP, SW, SP. More than 12%: GM, GC, SM, SC. 5–12%: Borderline cases require use of dual symbols	$C_u = \dfrac{D_{60}}{D_{10}}$ Greater than 4 $C_e = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Not meeting all gradation requirements for GW	
			Atterberg limits below 'A' line, or *PI* less than 4	Above 'A' line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Atterberg limits above 'A' line with *PI* greater than 7	
			$C_u = \dfrac{D_{60}}{D_{10}}$ Greater than 6 $C_e = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Not meeting all gradation requirements for SW	
			Atterberg limits below 'A' line with *PI* less than 5	Above 'A' line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Atterberg limits above 'A' line with *PI* greater than 7	

consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity. High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty, whereas a typical silt has the smooth feel of flour.

Toughness (consistency near plastic limit): After removing particles larger than the No. 40 sieve size, a specimen of soil about 1 cm^3 in size, is moulded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about 3 mm in diameter. The thread is then folded and re-rolled repeatedly. During this manipulation, the moisture content is gradully reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

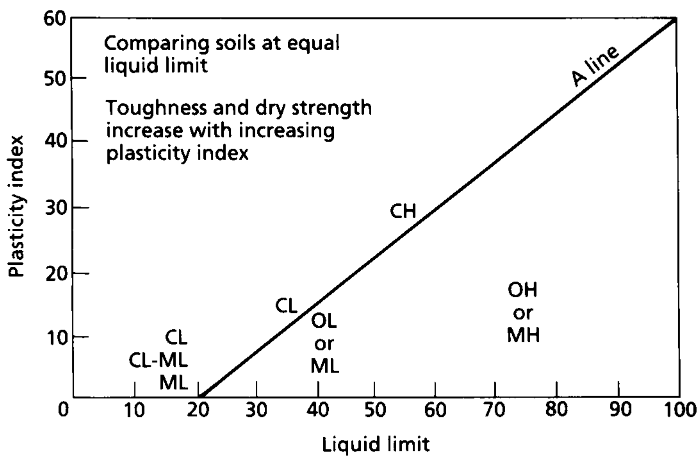
After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles. The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays that occur below the A-line. Highly organic clays have a very weak and spongy feel at the plastic limit.

Table 5.2.—Cont'd. Unified Soil Classification. Fine soils. More than half of the material is smaller than No. 200 sieve size[b]

Identification procedures on fraction smaller than No. 40 sieve size				Group symbols[a]	Typical names
	Dry strength* (crushing characteristics)	**Dilatancy* (reaction to shaking)**	**Toughness* (consistency near plastic limit)**		
Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity
	Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity
Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts micaceous or diatomaceous fine sandy or silty soils, clastic silts[b]
	High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays
	Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity
Highly organic soils	Readily identified by colour, odour, spongy feel, and frequently by fibrous texture			Pt	Peat and other highly organic soils

*See footnotes to Table

Table 5.2.—Cont'd.

Information required for describing soils		**Laboratory classification criteria**
Give typical name: indicate degree and character of plasticity, amount and maximum size of coarse grains, colour in wet conditions, odour if any, local or geological name, and other pertinent descriptive information and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: Clayey silt, brown: slightly plastic; small percentage of fine sand, numerous vertical root holes; firm and dry in place; loess; (ML)	Use grain-size curve in identifying the fractions as given under field identification	 Plasticity chart for laboratory classification of fine-grained soils

On this chart, the plasticity index is plotted against liquid limit. The A line is taken as the boundary between organic and inorganic soils, the latter lying above the line. Each of the main soil types and subgroups are given a letter, a pair of which are combined in the group symbol, the former being the prefix, the latter the suffix. Subsequently, the Unified Soil Classification system was developed from the Casagrande system.

According to Anon (1999), a full description of a soil should provide data on its particle size, plasticity, particle characteristics and colour, as well as its bedding, discontinuities and strength. Anon divides soils into coarse and fine types. Coarse soils are gravels and sands, and fine soils are silts and clays.

Boulders, cobbles, gravels, sands, silts and clays are distinguished as individual groups on the basis of their particle size distribution (Table 5.3a). Gravel, sand and silt have been subdivided into coarse-, medium- and fine-grained subgroups, and fine soils have been subdivided on the basis of plasticity. Coarse soils are described as well graded or poorly graded. Two further types of poorly graded coarse soils are recognized, namely, uniformly graded and gap graded (Fig. 1.18). Silts and clays are subdivided according to their liquid limits (Table 5.3b).

Most soils consist of more than one grade size type. If boulders and cobbles are present in composite soil types, then they are removed before an attempt is made at classification, their proportions being recorded separately. Their presence should be recorded in the soil description. Very coarse deposits should be described as boulders if over half of the very coarse material is of boulder size. They may be described as cobbly boulders if cobbles are an

Table 5.3a. Particle size distribution of soils

Types of material		Sizes (mm)
Boulders		Over 200
Cobbles		60–200
Gravel	Coarse	20–60
	Medium	6–20
	Fine	2–6
Sand	Coarse	0.6–2
	Medium	0.2–0.6
	Fine	0.06–0.2
Silt	Coarse	0.02–0.06
	Medium	0.006–0.02
	Fine	0.002–0.006
Clay		Less than 0.002

Table 5.3b. Plasticity according to liquid limit

Description	Plasticity	Range of liquid limit
Lean or silty	Low plasticity	Less than 35
Intermediate	Intermediate plasticity	35–50
Fat	High plasticity	50–70
Very fat	Very high plasticity	70–90
Extra fat	Extra high plasticity	Over 90

important second constituent in the very coarse fraction. If over half of the very coarse material is of cobble size, then it is described as cobbles. Similarly, it may be described as bouldery cobbles if boulders are an important second constituent in the very coarse fraction. Mixtures of the very coarse material and soil can be described by combining the terms for the very coarse constituent and the soil constituent as shown in Table 5.4.

According to Anon (1999), further factors that should be incorporated in a soil description, to help identification of soil, include the following:

1. Mass characteristics
   (a) Field strength or compactness and indication of moisture condition. A scale for estimating the strength of clays is given in Table 5.5, however, where assessment of strength is important, appropriate testing should be undertaken. The relative densities of sands and gravels may be determined by the standard penetration test (see Chapter 7) that, in turn, can be related to their angle of friction (Table 7.3).
   (b) The thickness of the bedding should be described as indicated in Table 2.1. Where beds are too thin to be described individually, they may be referred to as interbedded or interlaminated.
   (c) Discontinuities include joints, fissures, and shear surfaces. Their orientation, spacing, persistence, openness and surface texture (i.e. rough, smooth, polished, striated) should be described (see Chapter 2).

Table 5.4. Mixtures of very coarse materials and soil

Description	Composition
Boulders (or cobbles) with a little finer material*	Up to 5% finer material
Boulders (or cobbles) with some finer material*	5–20% finer material
Boulders (or cobbles) with much finer material*	20–50% finer material
Finer material* with many boulders (or cobbles)	50–20% boulders (or cobbles)
Finer material* with some boulders (or cobbles)	20–5% boulders (or cobbles)
Finer material* with occasional boulders (or cobbles)	Up to 5% boulders (or cobbles)

*Give the name of the finer material, e.g. Gravel with occasional boulders; cobbly boulders with some finer material (sand with some fines).

Table 5.5. Consistency of fine soils

Description	Consistency index (*IC*)	Approximate undrained shear strength (kPa)	Field identification
Hard	—	Over 300	Indented with difficulty by thumbnail, brittle
Very stiff	Above 1	150–300	Readily indented by thumbnail, still very tough
Stiff	0.75–1	75–150	Readily indented by thumb but penetrated only with difficulty. Cannot be moulded in the fingers
Firm	0.5–0.75	40–75	Can be penetrated several centimetres by thumb with moderate effort, and moulded in the fingers by strong pressure
Soft	Less than 0.5	20–40	Easily penetrated several centimetres by thumb, easily moulded
Very soft	—	Less than 20	Easily penetrated several centimetres by fist, exudes between fingers when squeezed in fist

$IC = \frac{LL - m}{LL - PL}$, where m = moisture content, PL = plastic limit and LL = liquid limit.

2. Material characteristics
   (a) The colour should relate to that of the overall impression of the soil. Soil with more than one colour can be described as mottled or multicoloured. A colour chart such as the Munsell chart should be used to help describe the colour of soils.
   (b) Particle shape, particle grading and composition. In particular instances, it may be necessary to describe the shape of soil particles (see Fig. 1.16). Grading of coarse soils has been referred to earlier.
   (c) Soil name (in capitals, e.g. SAND) is based on grading and plasticity. A coarse soil (omitting any boulders or cobbles) contains around 65% or more coarse material and is referred to as SAND or GRAVEL, according to the size fraction that predominates. Gravel and sand are further divided into coarse, medium and fine categories. Mixtures of coarse soil types can be described as shown in Table 5.6. Anon (1999) recommended that fine soil should be described as SILT or CLAY

Table 5.6. Mixed coarse soil types

Description	Composition of the coarse fraction
Slightly sandy gravel	Up to 5% sand
Sandy gravel	5–20% sand
Very sandy gravel	Over 20% sand
Gravel/sand	About equal proportions of gravel and sand
Very gravelly sand	Over 20% gravel
Gravelly sand	5–20% gravel
Slightly gravelly sand	Up to 5% gravel

depending on its plastic properties (although it cautioned against the use of the A-line on the plasticity chart as a reliable method of distinguishing between silts, which are supposed to plot below, and clays that plot above). It also was recommended that the terms should be mutually exclusive. In other words, terms such as silty clay were regarded as redundant. Field identification of fine soils can be made according to dilatancy, dry strength and toughness tests (see Table 5.2). The terms outlined in Table 5.7 can be used to describe common soils that include a mixture of soil types.

3. Geological formation, age and type of deposit.
4. Classification (optional).

When small amounts of organic matter occur throughout a soil, they can have a notable effect on plasticity and therefore the engineering properties. Increase in the quantities of organic matter can increase these effects. Nonetheless, soils in which the organic contents may be up to 30%, by weight, behave primarily as mineral soils.

Table 5.7. Description common types of soils

		Approximate proportion of secondary constituent	
Description	Main soil type	Coarse soil	Coarse or fine soil
Slightly clayey or silty and/or sandy or gravelly	Sand	—	>5%
Clayey or silty and/or sandy or gravelly	or	—	5–20%
Very clayey or silty and/or sandy or gravelly	Gravel	—	>20%
Very sandy or gravelly	Silt	>65%	
Sandy and/or gravelly	or	35– 65%	—
Slightly sandy and/or gravelly	Clay	<35%	

Table 5.8. Some values of gravels, sands and silts

	Gravels	Sands	Silts
Specific gravity	2.5–2.8	2.6–2.7	2.64–2.66
Bulk density (Mg m^{-3})	1.45–2.3	1.4–2.15	1.82–2.15
Dry density (Mg m^{-3})	1.4–2.1	1.35–1.9	1.45–1.95
Porosity (%)	20–50	23–35	—
Void ratio	—	—	0.35–0.85
Liquid limit (%)	—	—	24–35
Plastic limit (%)	—	—	14–25
Coefficient of consolidation (m^2 yr^{-1})	—	—	12.2
Cohesion (kPa)	—	—	75
Angle of friction (deg)	35–45	32–42	32–36

Peat is an accumulation of plant remains that has undergone some degree of decomposition. Inorganic soil material may occur as secondary constituents in peat, and should be described, for example, as slightly clayey or very sandy.

Coarse Soils

The microstructure of sand or gravel refers to its particle arrangement that, in turn, involves its packing. If grains approximate to spheres, then the closest type of systematic packing is rhombohedral packing, whereas the most open type is cubic packing, the porosities approximating to 26 and 48%, respectively. Put another way, the void ratio of a well-sorted and perfectly cohesionless aggregate of equidimensional grains can range between values of about 0.35 and 1.00. If the void ratio is more than unity, the microstructure will be collapsible or metastable. Some values of the physical properties of sands and gravels are given in Table 5.8.

Grain size and sorting have a significant influence on the engineering behaviour of coarse soils. Generally, the larger the particles, the higher the strength, and deposits consisting of a mixture of different-sized particles usually are stronger than those that are uniformly graded. For example, the amount of gravel in a sand–gravel mixture has a significant effect on shear strength, which increases considerably as the gravel content is increased up to 50 or 60%. Beyond this point, the material becomes less well graded, and the density does not increase. The density of a soil is governed by the manner in which its solid particles are packed. For instance, coarse soils may be densely or loosely packed. Densely packed sands are almost incompressible, whereas loosely packed deposits, located above the water table, are relatively compressible but otherwise stable. In other words, the behaviour of such sediments depends, to a large extent, on their relative density. Indeed, a maximum and minimum density can be distinguished. The smaller the range of particle sizes present and the more angular the particles, the smaller the minimum density. Conversely, if a wide range of particle sizes is present, the void space is reduced accordingly, hence the maximum density is higher. A useful way to characterize the density of a coarse-grained soil is by its relative density, D_r, which is defined as:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \tag{5.1}$$

where e is the naturally occurring void ratio, e_{max} is the maximum void ratio and e_{min} is the minimum void ratio. If the relative density of sand varies erratically, this can give rise to differential settlement. Generally, settlement in sands is relatively rapid. However, when the stresses are large enough to produce appreciable grain fracturing, there is a significant time lag.

Fundamentally, there are two basic mechanisms that contribute towards the deformation of coarse soil, namely, distortion of the particles and the relative motion between them. These two mechanisms usually are interdependent. At any instant during the deformation process, different mechanisms may be acting in different parts of the soil, and these may change as deformation continues. Interparticle sliding can occur at all stress levels, the stress required for its initiation increasing with decreasing void ratio. Crushing and fracturing of particles begins in a minor way at small stresses, becoming increasingly important when some critical stress is reached. This critical stress is smallest when the soil is loosely packed and uniformly graded, and consists of large angular particles with a low strength. Usually, fracturing only becomes important when the stress level exceeds 3.5 MPa.

The internal shearing resistance of a coarse soil is generated by friction when the grains in the zone of shearing are caused to slide, roll and rotate against each other. The angle of shearing resistance is influenced by the grain size distribution and grain shape. The larger the grains, the wider is the zone affected. The more angular the grains are, the greater the frictional resistance to their relative movement, since they interlock more thoroughly than do rounded ones. Therefore, they produce a larger angle of shearing resistance (Table 5.9).

Figure 5.1 shows that dense sand has a high peak strength and that when it is subjected to shear stress it expands up to the point of failure, after which a slight decrease in volume may occur. Conversely, loose sand compacts under shearing stress, and its residual strength may be similar to that of dense sand. Both curves in Figure 5.1 exhibit strains that are approximately

Table 5.9. Effect of grain shape and grading on the peak friction angle of sand

Shape and grading	Loose	Dense
Rounded, uniform	30°	37°
Rounded, well graded	34°	40°
Angular, uniform	35°	43°
Angular, well graded	39°	45°

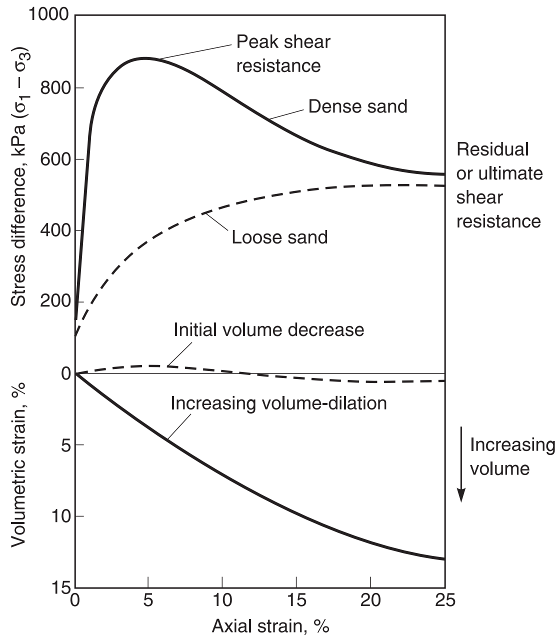


Figure 5.1

Stress–strain curves for dense and loose sand.

proportional to stress at low stress levels, suggesting a large component of elastic distortion. If the stress is reduced, the unloading stress–strain curve indicates that not all the strain is recovered on unloading. The hysteresis loss represents the energy lost in crushing and repositioning of grains. At higher shear stresses, the strains are proportionally greater, indicating greater crushing.

The presence of water in the voids of a coarse soil usually does not produce significant changes in the value of the angle of shearing resistance. However, if stresses develop in the pore water, they may bring about changes in the effective stresses between the particles, whereupon the shear strength and the stress–strain relationships may be altered radically.

Barton et al. (1993) distinguished between normal sands and those that had undergone a notable degree of diagenetic alteration. They regarded normal sands as those that did not possess any cohesion derived from grain interlock or cementation. Any cohesion possessed by these normal sands was due to the presence of a clay matrix. If such soils do not have a clay fraction, then they are cohesionless. Normal sands grade into diagenetically altered sands. Barton et al. recognized three groups of diagenetically altered sands, namely, locked sands, overlocked sands and slightly cemented sands. Locked sands show no visible bonding and, although trace amounts of cement may be present, their effect on strength is negligible. The cohesion of locked sands is derived from grain overgrowths. Cemented sands possess enough cement to develop cohesion but still break down into their component grains. At low stress levels, locked sands undergo high rates of dilation. Dilatancy becomes suppressed as the level of stress increases, since the asperities on the surfaces of individual grains are sheared through rather than causing dilation. They have peak frictional strengths considerably in excess of those of dense sand, with residual angles of friction varying between 30 and 35°.

Liquefaction is a phenomenon wherein a mass of soil loses a large percentage of its shear resistance and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear resistance. The basic cause responsible for the liquefaction of saturated sands is the build-up of excess pore water pressure due to either cyclic or shock loading of the sand. As a result, the grains of sand are compacted, with a consequent transfer of stress to the pore water and a reduction of stress on the sand grains. If drainage cannot take place, then the decrease in volume of the grains causes an increase in pore water pressure. If the pore water pressure builds up to the point where it is the same as the overburden pressure, then the effective stress is reduced to zero and the sand loses strength with a liquefied state developing. In loose sands the pore water pressure can increase rapidly to the value of the overburden or confining pressure. If the sand undergoes more or less unlimited deformation without mobilizing any notable resistance to deformation, then it can be described as having liquefied. However, Norris et al. (1998) pointed out that a loose sand does not lose all strength during liquefaction. Loose sands at low confining pressure, and medium and dense sands undergo only limited deformation due to dilation once initial liquefaction has occurred. Such response is referred to as ranging from limited liquefaction (in the case of loose and medium dense sands at low confining pressure) to dilative behaviour (in dense sands).

Silts and Loess

The grains in a deposit of silt often are rounded with smooth outlines. This influences their degree of packing. The latter, however, is more dependent on the grain size distribution within a silt deposit, uniformly sorted deposits not being able to achieve such close packing as those in which there is a range of grain size. This, in turn, influences the porosity and void ratio values, as well as bulk and dry densities (Table 5.8).

Dilatancy is characteristic of fine sands and silts. The environment is all important for the development of dilatancy since conditions must be such that expansion can take place. What is more, it has been suggested that the soil particles must be well wetted, and it appears that certain electrolytes exercise a dispersing effect, thereby aiding dilatancy. The moisture content at which a number of fine sands and silts from British formations become dilatant usually varies between 16 and 35%.

Consolidation of silt is influenced by grain size, particularly the size of the clay fraction, porosity and natural moisture content. Primary consolidation may account for over 75% of total consolidation. In addition, settlement may continue for several months after construction is completed because the rate at which water can drain from the voids under the influence of applied stress is slow.

The angle of shearing resistance decreases with increasing void ratio. It also is dependent on the plasticity index, grain interlocking and density.

Most loess is of aeolian origin. Wind-blown deposits of loess are characterized by a lack of stratification and uniform sorting, and occur as blanket deposits. Loess also is a remarkably uniform soil in terms of its dominant minerals. In other words, loess deposits have similar grain size distribution and mineral composition, as well as open texture, low degree of saturation, and bonding of grains that is not resistant to water. The fabric of loess takes the form of a loose skeleton built of grains (generally quartz) and micro-aggregates (assemblages of clay or clay and silty clay particles). The silt-sized particles are sub-angular and sub-rounded, and separate from each other, being connected by bonds and bridges, with uniformly distributed pores. The bridges are formed of clay-sized materials, be they clay minerals, fine quartz, feldspar or calcite. These clay-sized materials also occur as coatings to grains. Silica and iron oxide may be concentrated as cement at grain contacts, and amorphous overgrowths of silica occur on grains of quartz and feldspar. As silt-sized particles are not in contact, the mechanical behaviour of loess is governed by the structure and quality of the bonds.

Loess may exhibit sub-vertical columnar jointing. In addition, pipe systems may be developed in loess soils. Extensive pipe systems and sinkholes are present in some loess and have been referred to a loess karst. Pipes tend to develop by weathering and widening that takes place along the joint systems in loess. The depths to which pipes develop may be inhibited by changes in permeability associated with the occurrence of palaeosols.

Loess, as noted earlier, owes its engineering characteristics largely to the way in which it was deposited since this gives it a metastable structure, in that initially the particles are loosely packed. The porosity of the structure is enhanced by the presence of fossil root-holes. The latter are lined with carbonate cement, which helps bind the grains together. However, the chief binder is usually the clay matrix. On wetting, the clay bond in many loess soils becomes soft, which leads to the collapse of the metastable structure. The breakdown of the soil structure occurs under its own weight.

Loess deposits generally consist of 50–90% particles of silt size. In fact, sandy, silty and clayey loess can be distinguished (Fig. 5.2; Table 5.10). The range of dry density is very low to low (e.g. in Chinese loess, it may vary from 1.4 to 1.5 Mg m^{-3}). The low density is reflected

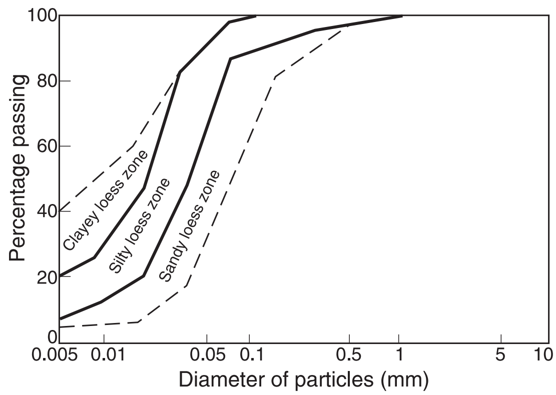


Figure 5.2

Particle size distribution of Missouri River Basin loess.

Table 5.10. Some geotechnical properties of loess soils

Property	Shaansi Province, China* Sandy loess	Shaansi Province, China* Clayey loess	Lanzhow Province, China+	Czechoslovakia, near Prague†	South Polish Uplands**
Natural moisture content (%)	9–13	13–20	11–10	21	3–26
Specific gravity					2.66–2.7
Bulk density (Mg m^{-3})	1.59–1.68	1.4–1.85			1.54–2.12
Dry density (Mg m^{-3})			1.4–1.5		1.46–1.73
Void ratio	0.8–0.92	0.76–1.11	1.05		
Porosity (%)				44–50	35–46
Grain-size distribution (%)					
Sand	20.5–35.2	12–15	20–24		
Silt	54.8–69.0	64–70	57–65		
Clay	8.0–15.5	17–24	16–21		
Plastic limit (%)			10–14	20	
Liquid limit (%)	26–28	30–31	27–30	36	
Plasticity index (%)	8–10	11–12	10–14	16	
Activity				1.32	
Coefficient of collapsibility	0.007–0.016	0.003–0.023		0.006–0.011	0.0002–0.06
Angle of friction					7–36°

*From Lin and Wang (1988).
+From Tan (1988).
†From Feda (1988).
**From Grabowska-Olszewska (1988).

in the void ratio and porosity. In the case of some Chinese loess, the void ratio varies from 0.81 to 0.89 and the porosity from 45–47%. Lutenegger and Hallberg (1988) observed that the bulk densities of unstable loess (such as the Peorian Loess, United States), tend to range between 1.34 and 1.55 Mg m^{-3}. If this material is wetted or consolidated (or reworked), the density increases, sometimes to as high as 1.6 Mg m^{-3} (Clevenger, 1958).

The liquid limit of loess averages about 30% (exceptionally, liquid limits as high as 45% have been recorded), and their plasticity index ranges from about 4 to 9%, but averages 6%. As far as their angle of shearing resistance is concerned, this usually varies from 30 to 34°. Loess deposits are better drained (their permeability ranges from 10^{-5} to 10^{-7} m s^{-1}) than are true silts because of the fossil root-holes. As would be expected, their permeability is appreciably higher in the vertical than in the horizontal direction.

Normally, loess possesses a high shearing resistance and can carry high loadings without significant settlement when natural moisture contents are low. For instance, moisture contents of undisturbed loess are generally around 10%, and the supporting capacity of loess at this moisture content is high. However, the density of loess is the most important factor controlling its shear strength and settlement. On wetting, large settlements and low shearing resistance are encountered when the density of loess is below 1.30 Mg m^{-3}, whereas if the density exceeds 1.45 Mg m^{-3}, settlement is small and shearing resistance fairly high.

Unlike silt, loess does not appear to be frost susceptible, this being due to its more permeable character, but it can exhibit quick conditions as with silt and it is difficult, if not impossible, to compact. Because of its porous structure, a "shrinkage" factor must be taken into account when estimating earthwork.

Several collapse criteria have been proposed that depend on the void ratios at the liquid limit, e_1, and the plastic limit, e_p, and the natural void ratio, e_o. Fookes and Best (1969) proposed a collapse index, i_c, which involved these void ratios, which is as follows:

$$i_c = \frac{e_o - e_p}{e_l - e_p} \qquad (5.2)$$

Previously, Feda (1966) had proposed the following collapse index:

$$i_c = \frac{m/S_r - PL}{PI} \qquad (5.3)$$

in which m is the natural moisture content, S_r is the degree of saturation, *PL* is the plastic limit and *PI* is the plasticity index. Feda also proposed that the soil must have a critical porosity of 40% or above and that an imposed load must be sufficiently high to cause structural collapse when the soil is wetted. He suggested that if the collapse index was greater than 0.85, then this was indicative of metastable soils. However, Northmore et al. (1996) suggested that a lower critical value of collapse index, that is 0.22, was more appropriate for some loess type soils in Essex, England. The double oedometer test also can be used to assess the degree of collapsibility. The test involves loading an undisturbed specimen at natural moisture content up to a given load. At this point, the specimen is flooded and the resulting collapse strain, if any, is recorded. Then, the specimen is subjected to further loading. The total consolidation upon flooding can be described in terms of the coefficient of collapsibility, C_{col}, given as:

$$C_{col} = \Delta h/h = \frac{\Delta e}{1+e} \tag{5.4}$$

in which Δh is the change in height of the specimen after flooding, h is the height of the specimen before flooding, Δe is the change in void ratio of the specimen upon flooding and e is the void ratio of the specimen prior to flooding. Table 5.11 provides an indication of the potential severity of collapse. This table indicates that those soils that undergo more than 1% collapse can be regarded as metastable. However, in China a figure of 1.5% is taken (Lin and Wang, 1988), and, in the United States, values exceeding 2% are regarded as indicative of soils susceptible to collapse (Lutenegger and Hallberg, 1988).

Clay Deposits

Clay deposits are composed principally of fine quartz and clay minerals. The three major clay minerals are kaolinite, illite and montmorillonite. Both kaolinite and illite have non-expansive lattices, whereas that of montmorillonite is expansive. In other words, montmorillonite is characterized by its ability to swell and by its notable cation exchange properties.

The microstructure of clay soils is governed largely by the clay minerals present and the forces acting between them. Because of the complex electrochemistry of clay minerals, the spatial arrangement of newly sedimented particles is influenced by the composition of the water in which deposition takes place. Single clay mineral platelets may associate in an edge-to-edge (EE), edge-to-face (EF), face-to-face (FF) or random type of arrangement, depending on the

Table 5.11. Collapse percentage as an indication of potential problems

Collapse (%)	Severity of problem
0–1	No problem
1–5	Moderate trouble
5–10	Trouble
10–20	Severe trouble
Above 20	Very severe trouble

interparticle balance between the forces of attraction and repulsion, and the amount or absence of turbulence in the water in which deposition occurs. The original microstructure of a clay deposit is modified subsequently by overburden pressures due to burial, which bring about consolidation. Consolidation tends to produce a preferred orientation with the degree of reorientation of clay particles being related to both the intensity of stress and the electrochemical environment, dispersion encouraging and flocculation discouraging clay particle parallelism. These microstructures are destroyed by weathering, gradually disappearing as the degree of weathering intensifies, as Coulthard and Bell (1993) found in the Lower Lias Clay in Gloucester, England.

The principal minerals in a deposit of clay tend to influence its index properties. For example, the plasticity of clay soil is influenced by the amount of its clay fraction and the type of clay minerals present since clay minerals influence the amount of attracted water held in a soil. Burnett and Fookes (1974), for instance, demonstrated that the clay fraction of the London Clay in the London Basin increases eastwards that, in turn, leads to an increase in its plasticity. Similarly, Bell (1994b) showed that high plasticity in the Speeton Clay, East Yorkshire, was influenced by the proportion of clay fraction present. Subsequently, Marsh and Greenwood (1995) noted that as the calcite content in the Gault Clay, England, increased, the liquid limit decreased. On the other hand, it would appear that there is only a general correlation between the clay mineral composition of a deposit and its activity. In other words, kaolinitic and illitic clays usually are inactive, whereas montmorillonitic clays range from inactive to active. Usually, active clays have a relatively high water-holding capacity and a high cation exchange capacity. They also are highly thixotropic, have low permeability and have low resistance to shear. The activity of clay was defined by Skempton (1953) as:

$$\text{Activity} = \frac{\text{Plasticity index}}{\text{Percentage by mass finer than 0.002 mm}} \tag{5.5}$$

He suggested three classes of activity, namely, active, normal and inactive, which he further subdivided into five groups as follows:

1. Inactive with activity less than 0.5,
2. Inactive with activity range 0.5–0.75,
3. Normal with activity range 0.75–1.25,
4. Active with activity range 1.25–2,
5. Active with activity greater than 2.

Particle size analyses of clay deposits indicate that they can contain appreciable fractions of grains larger than 0.002 mm. For example, Forster et al. (1994) reported that the clay fraction in the Gault Clay lay between 15 and 65% and that particles larger than 2 mm constituted less than 1%. The proportions of clay, silt and sand size material in the Claygate Beds and Bagshot Beds of south Essex, England, as recorded by Northmore et al. (1999), are given in Table 5.12.

Table 5.12. Particle size distribution in the clay deposits of the Claygate Beds and Bagshot Beds of South Essex (after Northmore et al.,1999)

	Clay (%)	Silt (%)	Sand (%)
Upper Claygate	50–57	40–42	3–8
Beds	53.5	41	5.5
Middle Claygate	51–54	36–46	3–10
Beds	52.8	42	5.2
Lower Claygate	52–61	36–45	2–3
Beds	55.2	42.3	2.5
Bagshot	53–69	28–41	1–15
Beds	61.2	35.2	3.6

The undrained shear strength is related to the amount and type of clay minerals present in a clay deposit, together with the presence of cementing agents. In particular, strength is reduced with increasing content of mixed-layer clay and montmorillonite in the clay fraction. The increasing presence of cementing agents, especially calcite, enhances the strength of the clay.

Geological age also has an influence on the engineering behaviour of a clay deposit. In particular, the porosity, moisture content and plasticity normally decrease in value with increasing depth and thereby age, whereas the strength and elastic modulus increase.

The engineering performance of clay deposits also is affected by the total moisture content and by the energy with which this moisture is held. For instance, the moisture content influences their consistency and strength, and the energy with which moisture is held influences their volume change characteristics. Indeed, one of the most notable characteristics of clays from the engineering point of view is their susceptibility to slow volume changes that can occur independently of loading due to swelling or shrinkage. Differences in the period and magnitude of precipitation and evapotransportation are the major factors influencing the swell–shrink response of active clay beneath a structure. These volume changes can give rise to ground movements that may result in damage to buildings. Low-rise buildings are particularly vulnerable to such ground movements since they generally do not have sufficient weight or strength to resist (Bell and Maud, 1995). These soils also represent a problem when they are encountered in road construction, and shrinkage settlement of embankments composed of such clays can lead to cracking and breaking up of the roads they support.

Grim (1952) distinguished two modes of swelling in clay soils, namely, intercrystalline and intracrystalline swelling. Intercrystalline swelling takes place when the uptake of moisture is restricted to the external crystal surfaces and the void spaces between the crystals. Intracrystalline swelling, on the other hand, is characteristic of the smectite family of clay

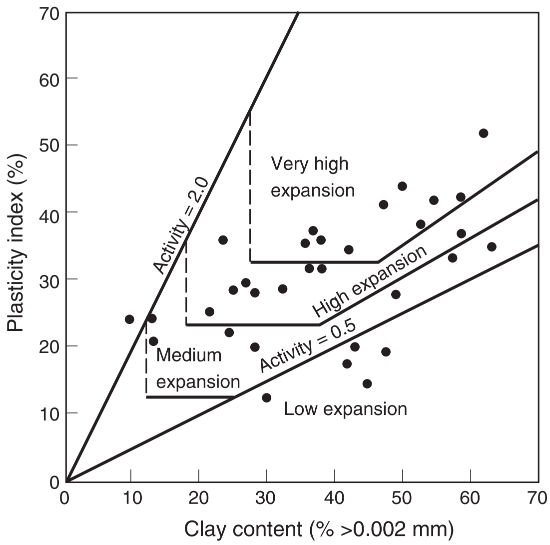


Figure 5.3

Use of the activity chart to estimate of the degree of expansiveness of some clay soils from Natal, South Africa (from Bell and Maud, 1995).

minerals, of montmorillonite in particular. The individual molecular layers that make up a crystal of montmorillonite are weakly bonded, so that on wetting moisture enters not only between the crystals but also between the unit layers that comprise the crystals. Generally, kaolinite has the smallest swelling capacity of the clay minerals, and nearly all of its swelling is of the intercrystalline type. Illite may swell by up to 15% but intermixed illite and montmorillonite may swell some 60–100%. Swelling in Ca montmorillonite is very much less than in the Na variety, it ranges from about 50–100%. Swelling in Na montmorillonite can amount to 2000% of the original volume, the clay then having formed a gel.

The maximum movement due to swelling beneath a building founded on expansive clay can be obtained from the following expression:

$$\text{Swell (\%)} = \frac{(PI - 10)}{10} \log_{10} S/p \tag{5.6}$$

where PI is the plasticity index, S is the soil suction at the time of construction (in kPa) and p is the overburden plus foundation pressure acting on each layer of soil (in kPa). One of the most widely used soil properties to predict swell potential is the activity of clay (Fig. 5.3).

The United States Army Engineers Waterways Experimental Station (USAEWES) classification of potential swell (Table 5.13) is based on the liquid limit (LL), plasticity index (PI) and initial (in situ) suction (S_i). The latter is measured in the field by a psychrometer.

Table 5.13. USAEWES classification of swell potential (After Snethan et al., 1977). With kind permission of the USAEWES

Liquid limit (%)	Plastic limit (%)	Initial (in situ) suction (kPa)	Potential swell (%)	Classification
Less than 50	Less than 25	Less than 145	Less than 0.5	Low
50–60	25–35	145–385	0.5–1.5	Marginal
Over 60	Over 35	Over 385	Over 1.5	High

The volume change that occurs due to evapotranspiration from a clay soil can be predicted conservatively by assuming the lower limit of the soil moisture content to be the shrinkage limit. Desiccation beyond this value cannot bring about further volume change. Transpiration from vegetative cover is a major cause of water loss from soils in semi-arid regions. Indeed, the distribution of soil suction in the soil is controlled primarily by transpiration from vegetation. The maximum soil suction that can be developed is governed by the ability of vegetation to extract moisture from the soil. The point, at which moisture is no longer available to plants is termed the permanent wilting point, and this corresponds to a pF value of about 4.2. The complete depth of active clay profiles usually does not become fully saturated during the wet season in semi-arid regions. Changes in soil suction may be expected over a depth of some 2.0 m between the wet and dry seasons. Swelling movements of over 350 mm have been reported for expansive clays in South Africa by Williams and Pidgeon (1983), and similar movements in similar soils have occurred in Texas.

The moisture characteristic (moisture content versus soil suction) of a soil provides valuable data concerning the moisture contents corresponding to the field capacity (defined in terms of soil suction, this is a pF value of about 2.0) and the permanent wilting point (pF of 4.2 and above), as well as the rate at which changes in soil suction take place with variations in moisture content. This enables an assessment to be made of the range of soil suction and moisture content that is likely to occur in the zone affected by seasonal changes in climate.

The extent to which the vegetation is able to increase the suction to the level associated with the shrinkage limit is important. In fact, the moisture content at the wilting point exceeds that of the shrinkage limit in soils with a high content of clay and is less in those possessing low clay contents. This explains why settlement resulting from the desiccating effects of trees is more notable in low to moderately expansive soils than in expansive ones. When vegetation is cleared from a site, its desiccating effect also is removed. Hence, the subsequent regain of moisture by clay soils leads to them swelling.

Desiccation cracks may extend to depths of 2 m in expansive clays and gape up to 150 mm. The suction pressure associated with the onset of cracking is approximately pF 4.6. The presence of desiccation cracks enhances evaporation from the soil. Such cracks lead to a variable development of suction pressure, the highest suction occurring nearest the cracks. This, in turn, influences the preconsolidation pressure as well as the shear strength. It has been claimed that the effect of desiccation on clay soils is similar to that of heavy overconsolidation.

Sridharan and Allam (1982), with reference to arid and semi-arid regions, found that repeated wetting and drying of clay soils can bring about aggregation of soil particles and cementation by compounds of Ca, Mg, Al and Fe. This enhances the permeability of the clays and increases their resistance to compression. Furthermore, interparticle desiccation bonding increases the shear strength, the aggregations offering higher resistance to stress. Indeed, depending on the degree of bonding, the expansiveness of an expansive clay soil may be reduced or it may even behave as a non-expansive soil.

Volume changes in clays also occur as a result of loading and unloading, which bring about consolidation and heave, respectively. When a load is applied to a clay soil, its volume is reduced, this principally being due to a reduction in the void ratio (Burland, 1990). If such a soil is saturated, then the load is carried initially by the pore water that causes a pressure, the hydrostatic excess pressure, to develop. The excess pressure of the pore water is dissipated at a rate that depends on the permeability of the soil mass, and the load is transferred eventually to the soil structure. The change in volume during consolidation is equal to the volume of the pore water expelled and corresponds to the change in void ratio of the soil. In other words, primary consolidation is brought about by a reduction in the void ratio. In clay soils, because of their low permeability, the rate of consolidation is slow. Further consolidation may occur due to a rearrangement of the soil particles. This secondary consolidation is usually much less significant. The compressibility of a clay soil is related to its geological history, that is, to whether it is normally consolidated or overconsolidated. A normally consolidated clay is one that at no time in its geological history has been subject to vertical pressure greater than its existing overburden pressure, whereas an overconsolidated clay has.

The compressibility of a clay soil can be expressed in terms of the compression index, C_c, or the coefficient of volume compressibility, m_v. The compression index tends to be applied to normally consolidated clays. The value of C_c for fine soils ranges from 0.075 for sandy clays to more than 1.0 for highly colloidal bentonic clays. An approximation of the degree of compressibility is given in Table 5.14a. It can be seen that the compressibility index increases with increasing clay content and so with increasing liquid limit. The coefficient of volume compressibility is defined as the volume change per unit volume per unit increase in load. The value of m_v for a given soil depends on the stress range over which it is determined. Anon, (1990a) recommended that it should be calculated for a pressure increment of 100 kPa in excess of the effective overburden pressure on the soil at the depth from which the sample was taken. Some typical values of m_v are given in Table 5.14b.

Table 5.14. Range of compressibility of fine soils

(a) Compressibility index

Soil type	Range (C_c)	Degree of compressibility
Soft clay	Over 0.3	Very high
Clay	0.3–0.15	High
Silt	0.15–0.075	Medium
Sandy clay	Less than 0.075	Low

(b) Some typical values of coefficient of volume compressibility

Coefficient of volume compressibility ($m^2\ MN^{-1}$)	Degree of compressibility	Soil types
Above 1.5	Very high	Organic alluvial clays and peats
0.3–1.5	High	Normally consolidated alluvial clays
0.1–0.3	Medium	Varved and laminated clays Firm to stiff clays
0.05–0.1	Low	Very stiff or hard clays Tills
Below 0.05	Very low	Heavily overconsolidated tills

When an excavation is made in clay with weak diagenetic bonds, elastic rebound causes immediate dissipation of some stored strain energy in the soil. However, part of the strain energy is retained due to the restriction on lateral straining in the plane parallel to the ground surface. The lateral effective stress either remains constant or decreases as a result of plastic deformation of the clay as time passes. This plastic deformation can result in significant time-dependent vertical heaving. However, creep of weakly bonded soils is not a common cause of heaving in excavations.

Overconsolidated clay is considerably stronger at a given pressure than normally consolidated clay, and it tends to dilate during shear, whereas normally consolidated clay consolidates (Burland, 1990). Hence, when overconsolidated clay is sheared under undrained conditions, negative pore water pressures are induced, the effective strength is increased, and the undrained strength is much higher than the drained strength, (the exact opposite to normally consolidated clay). When the negative pore water pressure gradually dissipates, the strength falls as much as 60 or 80% to the drained strength.

In 1964, Skempton observed that when clay is strained, it develops an increasing resistance (strength), but that under a given effective pressure, the resistance offered is limited, the maximum value corresponding to the peak strength. If testing is continued beyond the peak strength, then as displacement increases, the resistance decreases, again to a limiting value that is termed the residual strength. Skempton noted that in moving from peak to residual strength, cohesion falls to almost, or actually, zero and the angle of shearing resistance is reduced to a few degrees (it may be as much as 10° in some clays). He explained the drop in strength that occurred in overconsolidated clay as due to its expansion on passing peak strength and associated increasing water content on the one hand. On the other, he maintained that platey clay minerals became orientated in the direction of shear and thereby offered less resistance. Failure occurs once the stress on clay exceeds its peak strength and, as failure progresses, the strength of the clay along the shear surface is reduced to the residual value.

It was suggested that under a given effective pressure, the residual strength of clay is the same whether it is normal, or overconsolidated (Fig. 5.4). In other words, the residual shear strength of clay is independent of its post-depositional history, unlike the peak undrained shear strength that is controlled by the history of consolidation as well as diagenesis. Furthermore, the value of residual shear strength, ϕ_r', decreases as the amount of clay fraction increases in a deposit. In this context, not only is the proportion of detrital minerals important but so is that of the diagenetic minerals. The latter influence the degree of induration of a deposit of clay, and the value of ϕ_r' can fall significantly as the ratio of clay minerals to detrital and diagenetic minerals increases.

The shear strength of undisturbed clay is frequently found to be greater than that obtained when it is remoulded and tested under the same conditions and at the same water content levels. The ratio of the undisturbed to the remoulded strength at the same moisture content is termed the sensitivity of clay. Skempton and Northey (1952) proposed six grades of

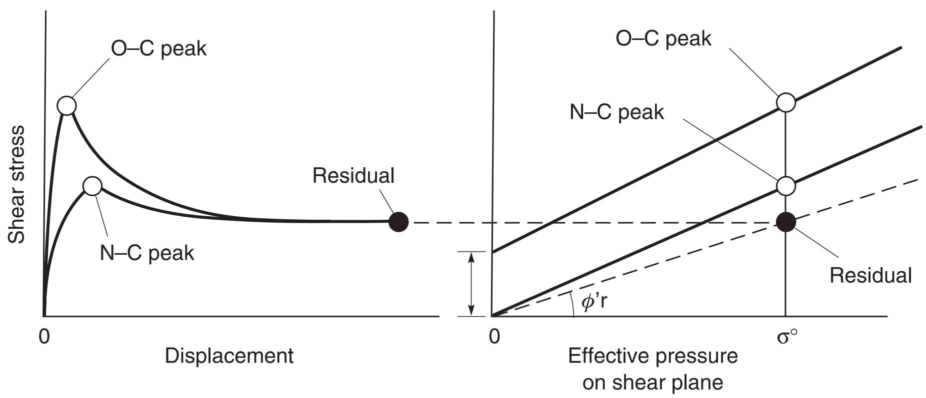


Figure 5.4

Peak strength and residual strength of normally consolidated (N–C) and overconsolidated (O–C) clay soils (after Skempton, 1964). With kind permission of the Institution of Civil Engineers.

sensitivity, namely, insensitive clays (under 1), low sensitive clays (1–2), medium sensitive clays (2–4), sensitive clays (4–8), extra-sensitive clays (8–16) and quick clay (over 16).

Clays with high sensitivity values have little or no strength after being disturbed. Indeed, if they suffer disturbance, this may cause an initially fairly strong material to behave as a viscous fluid. Any subsequent gain in strength due to thixotropic hardening does not exceed a small fraction of its original value. Sensitive clays generally possess high moisture contents, frequently with liquidity indices well in excess of unity. A sharp increase in moisture content may cause a great increase in sensitivity, sometimes with disastrous results. Heavily overconsolidated clays are insensitive.

Fissures play an extremely important role in the failure mechanism of overconsolidated clays. For example, the strength along fissures in clay is only slightly higher than the residual strength of the intact clay. Hence, it can be concluded that the upper limit of the strength of fissured clay is represented by its intact strength, whereas the lower limit corresponds to the strength along the fissures. The operational strength, which is somewhere between the two, however, is often significantly higher than the fissure strength. In addition to allowing clay to soften, fissures allow concentrations of shear stress that locally exceed the peak strength of clay, thereby giving rise to progressive failure. Under stress, the fissures in clay seem to propagate and coalesce in a complex manner. The ingress of water into fissures means that the pore water pressure in the clay concerned increases, which, in turn, means that its strength is reduced. Fissures in normally consolidated clays have no significant practical consequences.

The greatest variation in the engineering properties of clays can be attributed to the degree of weathering that they have undergone (Fig. 5.5). For instance, consolidation of a clay deposit gives rise to an anisotropic texture due to the rotation of the platey minerals. Secondly, diagenesis bonds particles together either by the development of cement, the intergrowth of adjacent grains or the action of van der Waals charges that are operative at very small grain separations. Weathering reverses these processes, altering the anisotropic structure and destroying or weakening interparticle bonding (Coulthard and Bell, 1993). Ultimately, weathering, through the destruction of interparticle bonds, leads to a clay deposit reverting to a normally consolidated, sensibly remoulded condition. Higher moisture contents are found in more weathered clay. This progressive degrading and softening also is accompanied by reductions in strength and deformation modulus with a general increase in plasticity. The reduction in strength has been illustrated by Cripps and Taylor (1981), who quoted the strength parameters for brown (weathered) and blue (unweathered) London Clay, as given in Table 5.15. These values indicate that the undrained shear strength, τ_u, is reduced by approximately half and that the effective cohesion, c', can suffer significant reduction on weathering. The effective angle of shearing resistance also is reduced and, at $\phi = 20°$, the value corresponds to a fully softened condition.

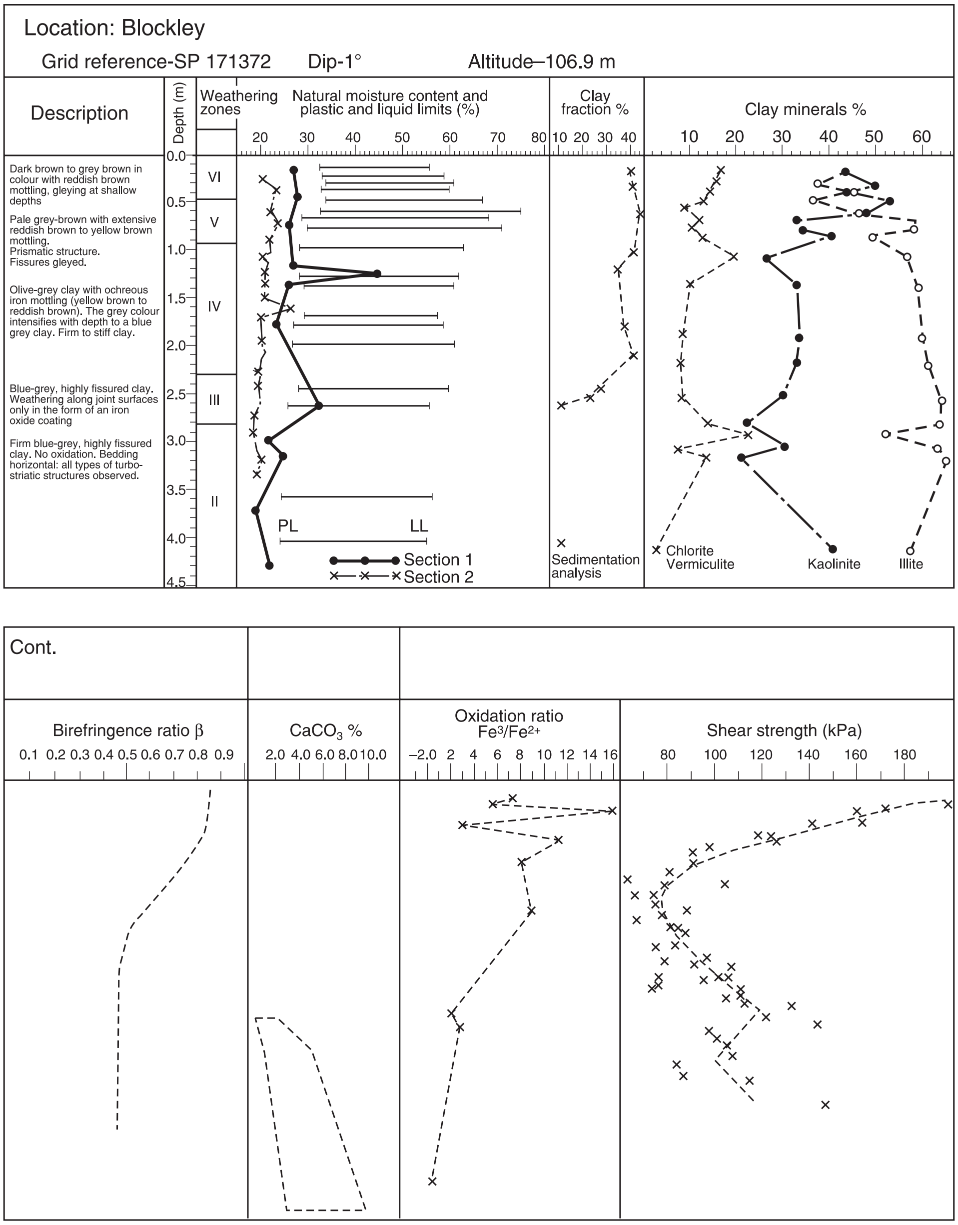


Figure 5.5

Geotechnical profile of the Lower Lias Clay at Blockley, Gloucestershire, England. A measure of the average orientation of clay particles seen in a thin section beneath the petrological microscope is afforded by the birefringence ratio (i.e. the ratio between the minimum and maximum light transmitted under crossed polars). This ratio varies between 0 for perfect parallel orientation to 1 for perfect random orientation. With increasing weathering, the birefringence ratio increases, indicating that the fabric of the clay becomes more disordered as the ground surface is approached. Weathering of pyrite (Fe_2S) produces sulphate and sulphuric acid. The latter reacts with calcium carbonate. The oxidation of iron compounds tends to increase as the surface is approached, which leads to increasing shear strength. *PL* = plastic limit; *LL* = liquid limit (after Coulthard and Bell, 1993).

Table 5.15. Strength of weathered (brown) and unweathered (blue) London Clay (after Cripps and Taylor, 1981)

Parameter	Brown	Blue
τ_u (kPa)	100–175	120–250
c' (kPa)	0–31	35–252
$\varphi°$	20–23	25–29

Tropical Soils

In humid tropical regions, weathering of rock is more intense and extends to greater depths than in other parts of the world. Residual soils develop in place as a consequence of weathering, primarily chemical weathering (Rahardjo et al., 2004). Consequently, climate (temperature and rainfall), parent rock, water movement (drainage and topography), age and vegetation cover are responsible for the development of the soil profile.

Ferruginous and aluminous clay soils are frequent products of weathering in tropical latitudes. They are characterized by the presence of iron and aluminium oxides and hydroxides. These compounds, especially those of iron, are responsible for the red, brown and yellow colours of the soils. The soils may be fine grained, or they may contain nodules or concretions. Concretions occur in the matrix where there are higher concentrations of oxides in the soil. More extensive accumulations of oxides give rise to laterite.

Laterite is a residual ferruginous clay-like deposit that generally occurs below a hardened ferruginous crust or hardpan (Charman, 1988). The ratios of silica (SiO_2) to sesquioxides (Fe_2O_3, Al_2O_3) in laterites usually are less than 1.33, those ratios between 1.33 and 2.0 are indicative of lateritic soils, and those greater than 2.0 are indicative of non-lateritic types. During drier periods, the water table is lowered. The small amount of iron that has been mobilized in the ferrous state by the groundwater is then oxidized, forming hematite, or goethite if hydrated. The movement of the water table leads to the gradual accumulation of iron oxides at a given horizon in the soil profile. A cemented layer of laterite is formed that may be a continuous or honeycombed mass, or nodules may be formed, as in laterite gravel. Concretionary layers often are developed near the surface in lowland areas because of the high water table.

Laterite hardens on exposure to air. Hardening may be due to a change in the hydration of iron and aluminium oxides.

Laterite commonly contains all size fractions from clay to gravel and sometimes even larger material (Fig. 5.6). Usually, at or near the surface, the liquid limit of laterite does not exceed 60% and the plasticity index is less than 30%. Consequently, laterite is of low to medium

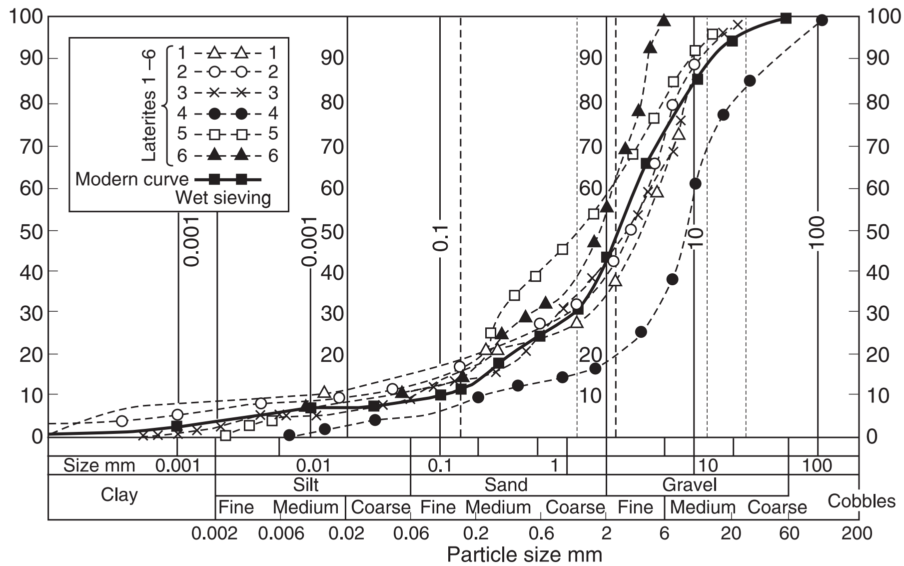


Figure 5.6

Grading curves of laterite (after Madu, 1977). With kind permission of Elsevier.

plasticity. The activity of laterite may vary between 0.5 and 1.75. Some values of common properties of laterite are given in Table 5.16.

Lateritic soils, particularly where they are mature, furnish a good bearing stratum (Blight, 1990). The hardened crust has a low compressibility and, therefore, settlement is likely to be negligible. In such instances, however, the strength of the soil may decrease with increasing depth.

Red earths or latosols are residual ferruginous soils in which oxidation readily occurs. Most such soils appear to have been derived from the first cycle of weathering of the parent material.

Table 5.16. Some common properties of laterites (after Madu, 1977)

Moisture content (%)	10–49
Liquid limit (%)	33–90
Plastic limit (%)	13–31
Clay fraction	15–45
Dry unit weight (kN m^{-3})	15.2–17.3
Cohesion (kPa)	466–782
Angle of friction (°)	28–35
Unconfined compressive strength (kPa)	220–825
Compression index	0.0186
Coefficient of consolidation (m^2 a^{-1})	262
Young's modulus (kPa)	5.63×10^4

They differ from laterite in that they behave as a clay soil and do not possess strong concretions. They, however, do grade into laterite.

Black clays typically are developed on poorly drained plains in regions with well-defined wet and dry seasons, where the annual rainfall is not less than 1250 mm. Generally, the clay fraction in these soils exceeds 50%, silty material varying between 20 and 40% and sand forming the remainder. The organic content usually is less than 2%. The liquid limits of black clays may range between 50 and 100%, with plasticity indices of between 25 and 70%. The shrinkage limit frequently is around 10–12%. Montmorillonite commonly is present in the clay fraction and is the chief factor determining the behaviour of these clays. For instance, they undergo appreciable volume changes on wetting and drying due to the montmorillonite content. These volume changes, however, tend to be confined to an upper critical zone of the soil, which frequently is less than 1.5 m thick. Below this, the moisture content remains more or less the same, for instance, around 25%.

Dispersive Soils

Dispersion occurs in soils when the repulsive forces between clay particles exceed the attractive forces, thus bringing about deflocculation so that, in the presence of relatively pure water, the particles repel each other to form colloidal suspensions. In non-dispersive soil, there is a definite threshold velocity below which flowing water causes no erosion. The individual particles cling to each other and are removed by water flowing with a certain erosive energy. By contrast, there is no threshold velocity for dispersive soil, the colloidal clay particles go into suspension even in quiet water and, therefore, these soils are highly susceptible to erosion and piping. Dispersive soils contain a moderate to high content of clay material but there are no significant differences in the clay fractions of dispersive and non-dispersive soils, except that soils with less than 10% clay particles may not have enough colloids to support dispersive piping. Dispersive soils contain a higher content of dissolved sodium (up to 12%) in their pore water than do ordinary soils. The clay particles in soils with high salt contents exist as aggregates and coatings around silt and sand particles, and the soil is flocculated. Dispersive soils generally occur in semi-arid regions where the annual rainfall is less than 860 mm (Bell and Walker, 2000).

For a given eroding fluid, the boundary between the flocculated and deflocculated states depends on the value of the sodium adsorption ratio. The sodium adsorption ratio, SAR, is used to quantify the role of sodium where free salts are present in the pore water and is defined as:

$$\mathrm{SAR} = \frac{\mathrm{Na}}{\sqrt{0.5\,(\mathrm{Ca} + \mathrm{Mg})}} \tag{5.7}$$

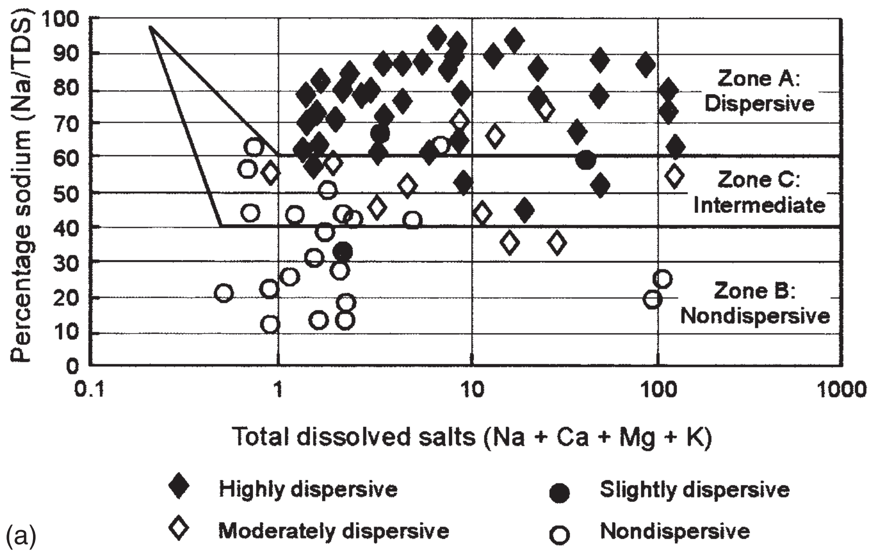
with units expressed in milli-equivalents per litre of the saturated extract. There is a relationship between the electrolyte concentration of the pore water and the exchangeable ions in the adsorbed layers of clay particles. This relationship is dependent on pH value and also may be influenced by the type of clay minerals present. Hence, it is not necessarily constant. Gerber and Harmse (1987) considered a SAR value greater than 10 as indicative of dispersive soils, between 6 and 10 as intermediate and less than 6 as nondispersive. However, Aitchison and Wood (1965) regarded soils in which the SAR exceeded 2 as dispersive.

Dispersive erosion depends on the mineralogy and chemistry of soil on the one hand, and the dissolved salts in the pore and eroding water on the other. The presence of exchangeable sodium is the main chemical factor contributing towards dispersive clay behaviour. This is expressed in terms of the exchangeable sodium percentage, ESP:

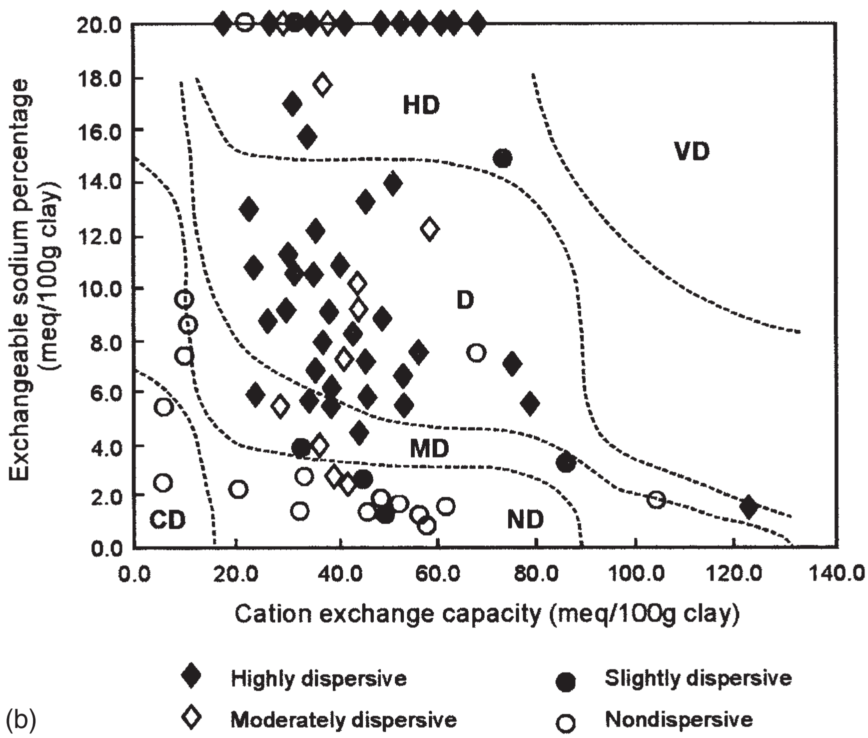
$$\mathrm{ESP} = \frac{\text{exchangeable sodium}}{\text{cation exchange capacity}} \times 100 \tag{5.8}$$

where the units are given in meq/100 g of dry soil. Above a threshold value of ESP of 10, soils have their free salts leached by seepage of relatively pure water and are prone to dispersion. Soils with ESP values above 15% are highly dispersive, according to Gerber and Harmse (1987). On the other hand, those soils with low cation exchange values (15 meq/100 g of clay) are non-dispersive at ESP values of 6% or below. Soils with high cation exchange capacity values and a plasticity index greater than 35% swell to such an extent that dispersion is not significant. High ESP values and piping potential may exist in soils in which the clay fraction is composed largely of smectitic and other 2:1 clays. Some illitic soils are highly dispersive. High values of ESP and high dispersibility are generally not common in clays composed largely of kaolinite.

Another property that has been claimed to govern the susceptibility of clayey soils to dispersion is the total content of dissolved salts, TDS, in the pore water. In other words, the lower the content of dissolved salts in the pore water, the greater the susceptibility of sodium-saturated clays to dispersion. Sherard et al. (1976) regarded the total dissolved salts for this specific purpose as the total content of calcium, magnesium, sodium and potassium in milli-equivalents per litre. They designed a chart in which sodium content was expressed as a percentage of TDS and was plotted against TDS to determine the dispersivity of soils (Fig. 5.7a). However, Craft and Acciardi (1984) showed that this chart had poor overall agreement with the results of physical tests. Furthermore, Bell and Maud (1994) showed that the use of the dispersivity chart to distinguish dispersive soils had not proved reliable in Natal, South Africa. There, the determination of dispersive potential frequently involves the use of a chart designed by Gerber and Harmse (1987) that plots ESP against cation exchange capacity, CEC (Fig. 5.7b).



(a)



(b)

Figure 5.7

(a) Potential dispersivity chart of Sherard et al., 1976, with some examples of soils from Natal, South Africa (after Bell and Walker, 2000). (b) Chart for classification of soils to determine their dispersivity, developed by Gerber and von Harmse (1987), with some examples of soils from Natal, South Africa (after Bell and Walker, 2000). The dispersivities plotted in (a) and (b) were determined by a rating system developed by a Bell and Walker.

Damage due to internal erosion of dispersive soil leads to the formation of pipes and internal cavities within slopes. Piping is initiated by dispersion of clay particles along desiccation cracks, fissures and root-holes. Piping has led to the failure of earth dams built with dispersive soil (Fig. 5.8). Indications of piping take the form of small leakages of muddy-coloured water after initial filling of the reservoir. In addition, severe erosion damage forming deep gullies occurs on embankments after rainfall. Fortunately, when dispersive soils are treated with lime, they are transformed to a non-dispersive state if the lime is mixed thoroughly into the soil.

Soils of Arid Regions

Most arid deposits consist of the products of physical weathering of bedrock formations. Weathering activity tends to be dominated by the physical breakdown of rock masses into poorly sorted assemblages of fragments ranging in size down to silts. Many of the deposits within alluvial plains and covering hillsides are poorly consolidated. As such, they may undergo large settlements, especially if subjected to vibration due to earthquakes or cyclic loading. Some gravels may consist of relatively weak, low-durability materials. Many arid areas are dominated by the presence of large masses of sand. Depending on the rate of

Figure 5.8

Failure of a small dam constructed of dispersive soil, near Ramsgate, Natal, South Africa.

supply of sand, the wind speed, direction, frequency and constancy and the nature of the ground surface, sand may be transported and/or deposited in mobile or static dunes. For the most part, aeolian sands are poorly (uniformly) graded. In the absence of downward leaching, surface deposits become contaminated with precipitated salts, particularly sulphates and chlorides. Alluvial plain deposits often contain gypsum particles and cement, and also fragments of weak weathered rock and clay.

In arid regions, sabkha conditions commonly develop in low-lying coastal zones and inland plains with shallow water tables. These are extensive saline flats that are underlain by sand, silt or clay and often are encrusted with salt. Highly developed sabkhas tend to retain a greater proportion of soil moisture than moderately developed sabkhas. Within coastal sabkhas, the dominant minerals are calcite ($CaCO_3$), dolomite [$(Ca,Mg)CO_3$] and gypsum ($CaSO_4.nH_2O$), with lesser amounts of anhydrite ($CaSO_4$), magnesite ($MgCO_3$), halite (NaCl) and carnalite ($KCl.MgCl_2.6H_2O$), together with various other sulphates and chlorides (James and Little, 1994). Highly saline groundwater may contain up to 23% sodium chloride and occur close to ground level. In fact, the sodium chloride content of groundwater can be high enough to represent a corrosion hazard.

Minerals that are precipitated from groundwater in arid deposits also have high solution rates, so that flowing groundwater may lead to the development of solution features. Problems such as increased permeability, reduced density and settlement are liable to be associated with engineering works or natural processes that result in a decrease in the salt concentration of groundwater. Changes in the state of hydration of minerals, such as swelling clays and calcium sulphate, also cause significant volume changes in soils. In particular, low-density sands that are cemented with soluble salts such as sodium chloride are vulnerable to salt removal by dissolution by freshwater, leading to settlement. Hence, rainstorms and burst water mains present a hazard, as does watering of grassed areas and flower beds. The latter should be controlled, and major structures should be protected by drainage measures to reduce the risks associated with rainstorms or burst water pipes. In the case of inland sabkhas, the minerals precipitated within the soil are much more variable than those of coastal sabkhas since they depend on the composition of local groundwater.

Sabkha soils frequently are characterized by low strength. Furthermore, some surface clays that are normally consolidated or lightly overconsolidated may be sensitive to highly sensitive. The low strength is attributable to the concentrated salt solutions in sabkha brines; the severe climatic conditions under which sabkha deposits are formed (e.g. large variations in temperature and excessive wetting–drying cycles) that can give rise to instability in sabkha soils; and the ready solubility of some of the minerals that act as cements in these soils. As a consequence, the bearing capacity of sabkha soils and their compressibility frequently do not meet routine design requirements.

A number of silty deposits formed under arid conditions are liable to undergo considerable volume reduction or collapse when wetted. Such metastability arises due to the loss of strength of interparticle bonds, resulting from increases in water content. Thus, infiltration of surface water, including that applied by irrigation, leakage from pipes and rise of the water table may cause large settlements to occur.

A common feature of arid regions is the cementation of sediments by the precipitation of mineral matter from the groundwater. The species of salt held in solution, and also those precipitated, depends on the source of the water, as well as the prevailing temperature and humidity conditions. The process may lead to the development of various crusts or cretes in which unconsolidated deposits are cemented. The most commonly precipitated material is calcium carbonate (Netterburg, 1994). As the carbonate content increases in these soils, it first occurs as scattered concentrations of flaky habit, then as hard concretions. Once it exceeds 60%, the concentration becomes continuous. These deposits are referred to as calcrete (Fig. 5.9). The calcium carbonate in calcrete profiles decreases from top to base, as generally does the hardness. The development of calcrete is inhibited beyond a certain aridity since the low precipitation is unable to dissolve and drain calcium carbonate towards the water table. Consequently, in very arid climates, gypcrete may take the place of calcrete.

Figure 5.9

Calcrete in the north of Namib-Nauluft Park, Namibia.

Tills and Other Glacially Associated Deposits

Till usually is regarded as being synonymous with boulder clay. It is deposited directly by ice, whereas stratified drift is deposited in melt waters associated with glaciers. The character of till deposits varies appreciably and depends on the lithology of the material from which it was derived, on the position in which it was transported in the glacier, and on the mode of deposition. The underlying bedrock material usually constitutes up to about 80% of basal or lodgement tills, depending on its resistance to abrasion and plucking.

Deposits of till consist of a variable assortment of rock debris ranging from fine rock flour to boulders (Hughes et al., 1998). The shape of the rock fragments found in till varies but is conditioned largely by their initial shape at the moment of incorporation into the ice. Angular boulders are common, their irregular sharp edges resulting from crushing. Tills may consist essentially of sand and gravel with very little binder, alternatively they may have an excess of clay. Lenses and pockets of sand, gravel and highly plastic clay frequently are encountered in some tills. Argillaceous rocks, such as shales and mudstones, are abraded more easily and so produce fine-grained tills that are richer in clay minerals and, therefore, more plastic than other tills. Mineral composition also influences the natural moisture content, which is slightly higher in tills containing appreciable quantities of clay minerals.

Lodgement till is plastered on to the ground beneath a moving glacier in small increments as the basal ice melts. Because of the overlying weight of ice, such deposits are overconsolidated. Due to abrasion and grinding, the proportion of silt and clay size material is relatively high in lodgement till (e.g. the clay fraction varies from 15 to 40%). Lodgement till is commonly stiff, dense and relatively incompressible (Sladen and Wrigley, 1983). Hence, it is practically impermeable. Fissures frequently are present in lodgement till, especially if it is clay matrix dominated.

Ablation till accumulates on the surface of the ice when englacial debris melts out, and as the glacier decays, the ablation till is lowered slowly to the ground. Therefore, it is normally consolidated. Ablation tills have a high proportion of far-travelled material and may not contain any of the local bedrock. Because it has not been subjected to much abrasion, ablation till is characterized by abundant large fragments that are angular and not striated, the proportion of sand and gravel is high and clay is present only in small amounts (usually less than 10%). Because the texture is loose, ablation till can have an extremely low in situ density. Since ablation till consists of the load carried at the time of ablation, it usually forms a thinner deposit than lodgement till.

The particle size distribution and fabric (stone orientation, layering, fissuring and jointing) are among the most significant features as far as the engineering behaviour of a till is concerned. McGown and Derbyshire (1977) therefore used the percentage of fines to distinguish granular, well-graded and matrix-dominated tills, the boundaries being placed at 15 and 45%, respectively.

Tills frequently are gap graded, the gap generally occurring in the sand fraction (Fig. 5.10). Large, often very local, variations can occur in the grading of till that reflect local variations in the formation processes, particularly the comminution processes. The range in the proportions of coarse and fine fractions in tills dictates the degree to which the properties of the fine fraction influence the properties of the composite soil. The variation in the engineering properties of the fine soil fraction is greater than that of the coarse fraction, and this often tends to dominate the engineering behaviour of the till.

The specific gravity of till deposits often is remarkably uniform, varying from 2.77 to 2.78. These values suggest the presence of fresh minerals in the fine fraction, that is, rock flour rather than clay minerals. Rock flour behaves more like granular material than cohesive and has a low plasticity. The consistency limits of tills are dependent on moisture content, grain size distribution and the properties of the fine-grained fraction (Bell, 2002). Generally, however, the plasticity index is small and the liquid limit of tills decreases with increasing grain size. The variations in some simple index properties with depth of the Upper Boulder Clay of Teesside, England, are given in Figure 5.11.

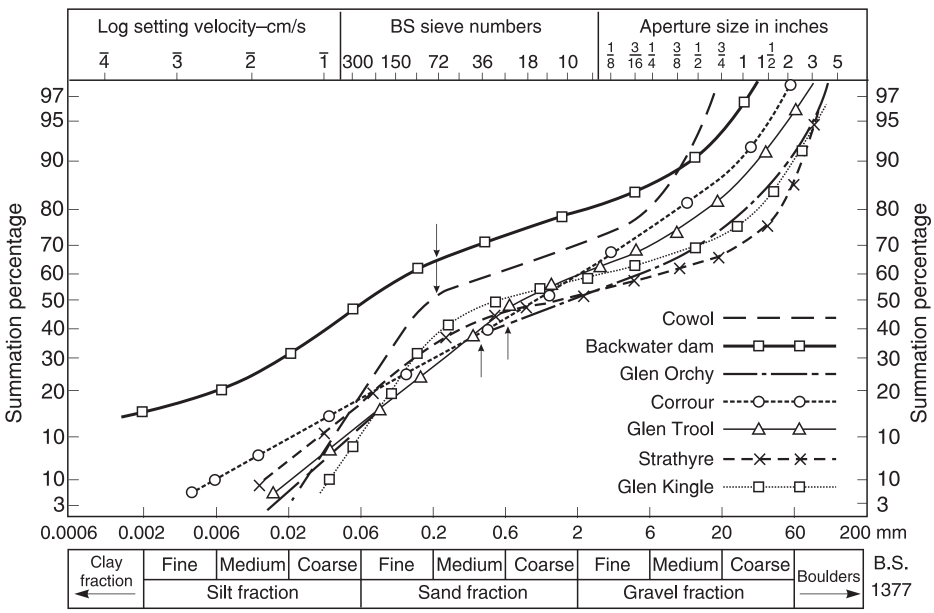


Figure 5.10

Typical gradings of some Scottish tills (after McGown, 1971). With kind permission of the Geological Society.

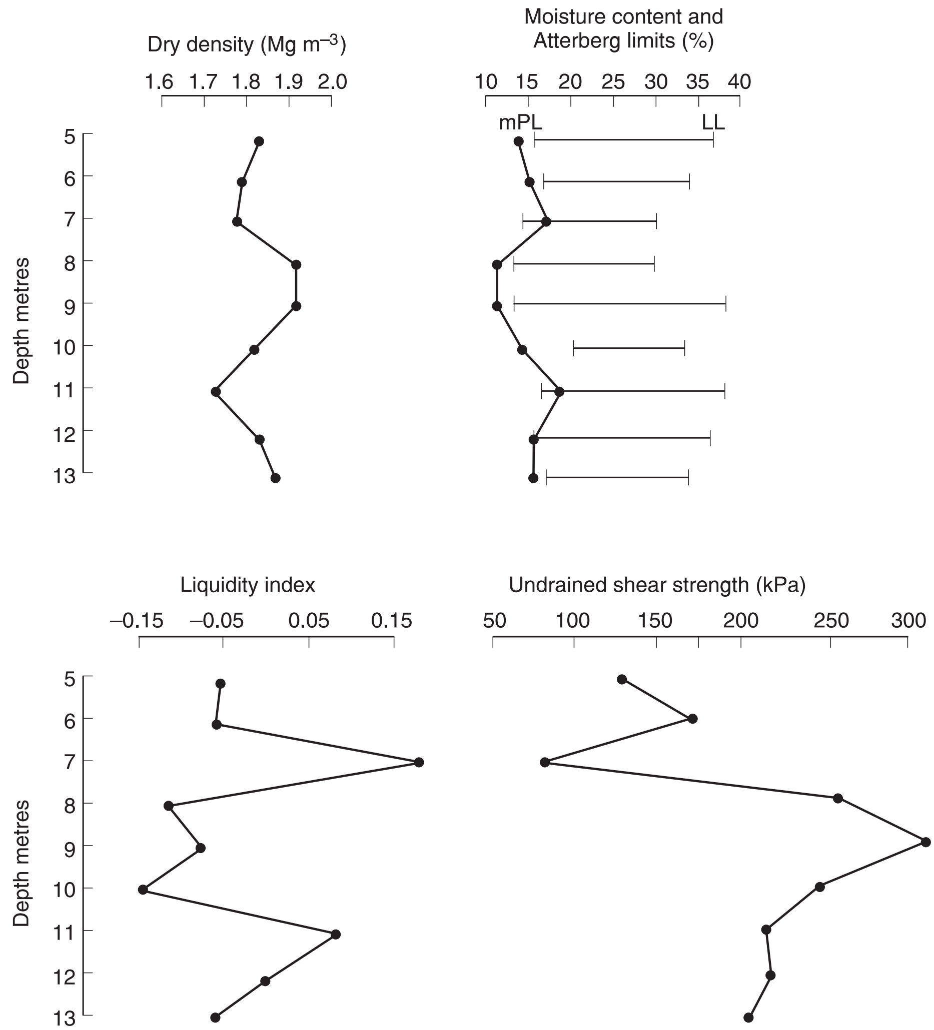


Figure 5.11

Variation in some simple index properties with depth of the Upper Boulder Clay of Teesside, England (after Bell, 2002).

The compressibility and consolidation of tills are determined principally by the clay content, as is the shear strength (Table 5.17). For example, the value of compressibility index tends to increase linearly with increasing clay content, whereas for tills of very low clay content (less than 2%), this index remains about constant ($C_c = 0.01$). The shear strength of till can range from 150 kPa to over 1.5 MPa.

Fissures in till tend to be variable in character, spacing, orientation and areal extent, although they can have a preferred orientation. Opening up and softening along these fissures gives

Table 5.17. Strength of tills from Holderness (after Bell, 2002)

	Unconfined compressive strength (kPa)		Sensitivity	Direct shear				Triaxial			
	Intact	Remoulded		c	$\phi°$	c_r	$\phi_r°$	c_u	$\phi_u°$	c'	$\phi°$
1. Hessle Till (Dimlington, Hornsea)											
Max	138	116	1.31 (L)	30	25	3	23	98	8	80	24
Min	96	74	1.10 (L)	16	16	0	13	22	5	10	13
Mean	106	96	1.19 (L)	20	24	1	20	35	7	26	25
2. Withernsea Till (Dimlington)											
Max	172	148	1.18 (L)	38	30	2	27	62	19	42	34
Min	140	122	1.15 (L)	21	20	0	18	17	5	17	16
Mean	160	136	1.16 (L)	26	24	1	21	30	9	23	25
3. Skipsea Till (Dimlington)											
Max	194	168	1.15 (L)	45	38	5	35	50	21	25	36
Min	182	154	1.08 (L)	25	20	0	19	17	10	22	24
Mean	186	164	1.13 (L)	27	26	1	25	29	12	28	30
4. Basement Till (Dimlington)											
Max	212	168	1.27 (L)	47	34	2	30	59	17	42	36
Min	163	140	1.19 (L)	23	20	0	18	22	6	19	20
Mean	186	156	1.21 (L)	29	24	1	23	38	9	34	29

Note: c = cohesion in kPa, r = residual, u = undrained; ϕ = angle of friction; L = low sensitivity.

rise to a rapid reduction of undrained shear strength along the fissures. In fact, the undrained shear strength along fissures in till may be as little as one-sixth that of the intact soil. The nature of the various fissure coatings (sand, silt or clay-size material) is of critical importance in determining the shear strength behaviour of fissured tills. Deformation and permeability also are controlled by the nature of the fissure surfaces and coatings.

Eyles and Sladen (1981) recognized four zones of weathering within the soil profile of lodgement till in the coastal area of Northumberland, England (Table 5.18a). As the degree of weathering of the till increases, so does the clay fraction and moisture content. This, in turn, leads to changes in the liquid and plastic limits and in the shear strength (Table 5.18b and Fig. 5.12).

Deposits of stratified drift often are subdivided into two categories, namely, those that develop in contact with ice, namely, ice contact deposits, and those that accumulate beyond the limits of ice, forming in streams, lakes or seas, that is, proglacial deposits.

Table 5.18a. A weathering scheme for Northumberland lodgement tills (after Eyles and Sladen, 1981). With kind permission of Elsevier

Weathering zone	Zone	Description	Maximum depth (m)
Highly weathered	IV	Oxidized till and surficial material Strong oxidation colours High rotten boulder content Leaching of most primary carbonate Prismatic gleyed jointing Pedological profile usually leached brown earth	3
Moderately weathered	III	Oxidized till Increased clay content Low rotten boulder content Little leaching of primary carbonate Usually dark brown or dark red brown Base commonly defined by fluvioglacial sediments	8
Slightly weathered	II	Selective oxidation along fissure surfaces where present, otherwise as Zone I	10
Unweathered	I	Unweathered till No post-depositionally rotted boulders No oxidation No leaching of primary carbonate Usually dark grey	

Table 5.18b. Typical geotechnical properties for Northumberland lodgement tills (after Eyles and Sladen, 1981). With kind permission of Elsevier

	Weathered zones	
Property	**I**	**III & IV**
Bulk density (Mg m^{-3})	2.15–2.30	1.90–2.20
Natural moisture content (%)	10–15	12–25
Liquid limit (%)	25–40	35–60
Plastic limit (%)	12–20	15–25
Plasticity index	0–20	15–40
Liquidity index	–0.20 to –0.05	III –0.15 to +0.05 IV –0 to +30
Grading of fine (<2 mm) fraction		
% clay	20–35	30–50
% silt	30–40	30–50
% sand	30–50	10–25
Average activity	0.64	0.68
c' (kPa)	0–15	0–25
ϕ' (degrees)	32–37	27–35
ϕ_r' (degrees)	30–32	15–32

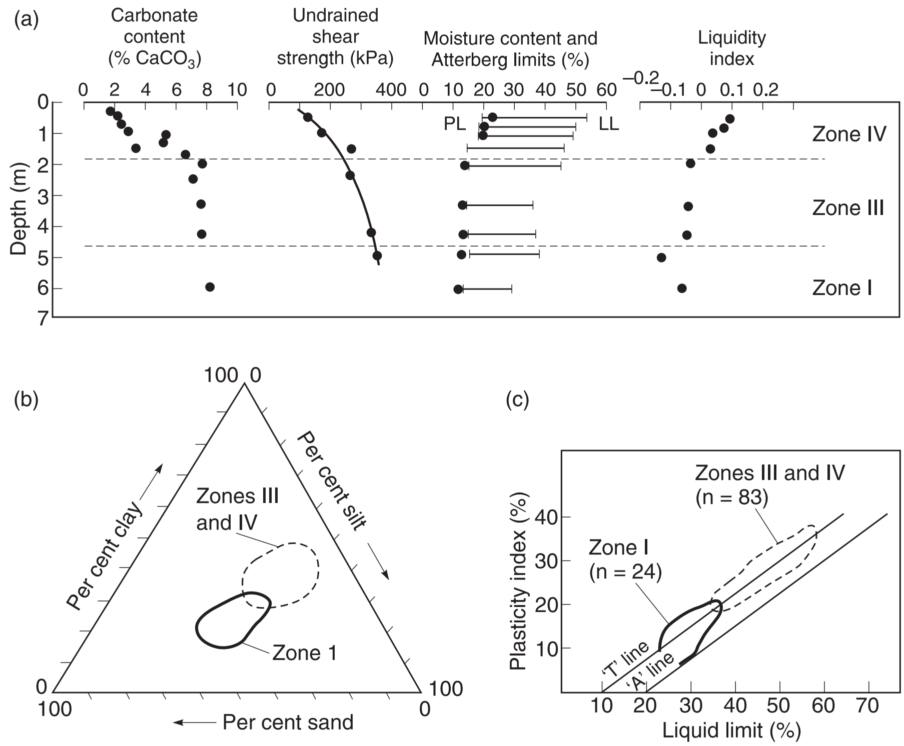


Figure 5.12

(a) Lodgement till from Northumberland, England, showing the variation with depth of carbonate content, undrained shear strength, moisture content, Atterberg limits and liquidity index. (b) Particle size distribution for weathered and unweathered tills shown in (a). (c) Plasticity chart for weathered and unweathered tills shown in (a). *n* = number of determinations (after Eyles and Sladen, 1981). With kind permission of Elsevier.

The range of particle size found in outwash fans varies from coarse sands to boulders. When they are first deposited, their porosity may be anything from 25 to 50%, and they tend to be very permeable. The finer silt–clay fraction tends to be transported further downstream. Other ice contact deposits, namely, kames, kame terraces and eskers, usually consist of sands and gravels.

The most familiar proglacial deposits are varved clays. The thickness of the individual varve is frequently less than 2 mm, although much thicker layers have been noted in some deposits. Generally, the coarser layer is of silt size and the finer of clay size. Varved clays tend to be normally consolidated or lightly overconsolidated, although it usually is difficult to make the distinction. In many cases, the precompression may have been due to ice loading. The range of liquid limit for varved clays tends to vary between 30 and 80%, whereas that of plastic limit often varies between 15 and 30% (Table 5.19). These limits allow the material to

Table 5.19. Some properties of varved and laminated clays

	Varved clays, Elk Valley, British Columbia*	Laminated clays, Teesside, England**
Moisture content (%)	35	25–35 (30)
Plastic limit (%)	22	18–31 (26)
Liquid limit (%)	34	29–78 (56)
Plasticity index (%)	15.5	19–49 (33)
Liquidity index	0.36	−0.12–0.35 (0.15)
Activity	0.36	0.47–0.65 (0.54)
Linear shrinkage		9–14 (11)
Compression index	0.405–0.587 (0.496)	0.55
Undrained shear strength (kPa)		20–102 (62)

Note: Range with average value in brackets.
*After George, 1986.
**After Bell and Coulthard, 1997.

be classified as inorganic silty clay of medium to high plasticity or compressibility. In some varved clay, the natural moisture content is near the liquid limit. Consequently, these clays are soft and frequently have sensitivities around 4 (Bell and Coulthard, 1997). Their activity tends to range between active and normal, and some may be expansive. The average strength of some varved clays, for example, from Ontario, is about 40 kPa, with a range of 24–49 kPa. The effective stress parameters of apparent cohesion and angle of shearing resistance range from 0.7 to 19.5 kPa, and 22–25°, respectively.

The material of which quick clays are composed is predominantly smaller than 0.002 mm (Geertsema and Torrance, 2005). Many deposits, however, seem to be very poor in clay minerals, containing a high proportion of ground-down, fine quartz. The fabric of these soils contains aggregations. Granular particles, whether aggregations or primary minerals, are rarely in direct contact, being linked generally by bridges of fine particles. Clay minerals usually are non-oriented, and clay coatings on primary minerals tend to be uncommon, as are cemented junctions. Networks of platelets occur in some soils. Quick clays generally exhibit little plasticity, their plasticity index usually varying from 8 to 12%. Their liquidity index normally exceeds 1, and their liquid limits are often less than 40%. Quick clays usually are inactive, their activity frequently being less than 0.5. The most extraordinary property possessed by quick clays is their very high sensitivity. In other words, a large proportion of their undisturbed strength is permanently lost following shear (Fig. 5.13). The small fraction of the original strength regained after remoulding may be attributable to the development of some different form of interparticle bonding. The reason why only a small fraction of the original strength is recovered is because the rate at which it develops is so slow.

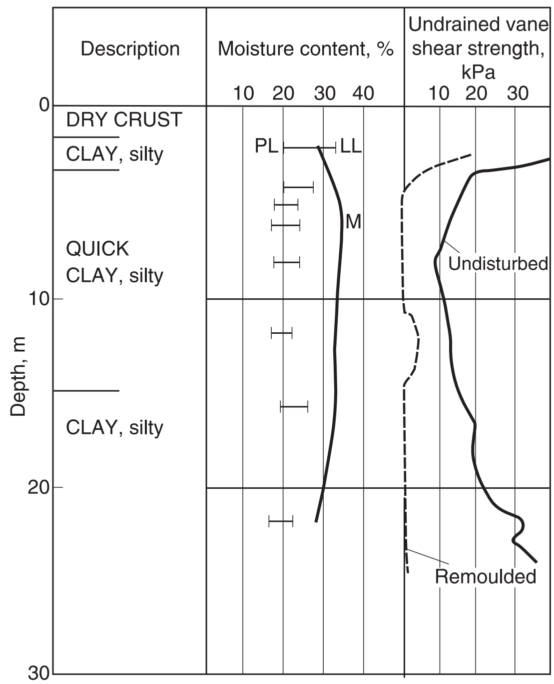


Figure 5.13

Moisture content, consistency indices, undrained shear strength and sensitivity of quick clay from near Trondheim, Norway (from Bell and De Bruyn, 1998).

Frost Action in Soil

Frost action in a soil is influenced by the initial temperature of the soil, as well as the air temperature; the intensity and duration of the freeze period; the depth of frost penetration; the depth of the water table; and the type of ground cover. If frost penetrates down to the capillary fringe in fine soils, especially silts, then, under certain conditions, lenses of ice may be developed. The formation of such ice lenses may, in turn, cause frost heave and frost boil that may lead to the break-up of roads, the failure of slopes, etc. Shrinkage, which is attributable to thermal contraction and desiccation, gives rise to polygonal cracking in the ground. Water that accumulates in the cracks is frozen and consequently helps increase their size. This action may lead to the development of lenses of ice.

Classification of Frozen Soil

Ice may occur in frozen soil as small disseminated crystals whose total mass exceeds that of the mineral grains. It also may occur as large tabular masses that range up to several metres thick, or as ice wedges. The latter may be several metres wide and may extend to 10 m or so in depth. As a consequence, frozen soils need to be described and classified for engineering purposes. A method of classifying frozen soils involves the identification of the soil type and the character of the ice (Andersland, 1987). First, the character of the actual soil is classified according to the Unified Soil Classification system (Table 5.2). Second, the soil characteristics consequent on freezing are added to the description. Frozen-soil characteristics are divided into two basic groups based on whether or not segregated ice can be seen with the naked eye (Table 5.20). Third, the ice present in the frozen soil is classified; this refers to inclusions of ice that exceed 25 mm in thickness.

The amount of segregated ice in a frozen mass of soil depends largely on the intensity and rate of freezing. When freezing takes place quickly, no layers of ice are visible, whereas slow

Table 5.20. Description and classification of frozen soils (from Andersland, 1987)

I. Description of soil phase (independent of frozen state)	**Classify soil phase by the Unified Soil Classification system**					
	Major group		Subgroup			
	Description	Designation	Description		Designation	
	Segregated ice not visible by eye	N	Poorly bonded or friable		Nf	
				No excess ice	Nb	n
			Well bonded	Excess ice		e
II. Description of frozen soil			Individual ice crystals or inclusions		Vx	
	Segregated ice visible by eye (ice 25 mm or less thick)	V	Ice coatings on particles		Ve	
			Random or irregularly oriented ice formations		Vr	
			Stratified or distinctly oriented ice formations		Vs	
III. Descriptionof substantial ice strata	Ice greater than 25 mm thick	ICE	Ice with soil inclusions		ICE + soil type	
			Ice without soil inclusions		ICE	

freezing produces visible layers of ice of various thicknesses. Ice segregation in soil also takes place under cyclic freezing and thawing conditions.

Mechanical Properties of Frozen Soil

The presence of masses of ice in a soil means that as far as engineering is concerned, the properties of both have to be taken into account. Ice has no long-term strength, that is, it flows under very small loads. If a constant load is applied to a specimen of ice, instantaneous elastic deformation occurs. This is followed by creep, which eventually develops a steady state. Instantaneous elastic recovery takes place on removal of the load, followed by recovery of the transient creep.

The relative density influences the behaviour of frozen coarse soils, especially their shearing resistance, in a manner similar to that when they are unfrozen. The cohesive effects of the ice matrix are superimposed on the latter behaviour, and the initial deformation of frozen sand is dominated by the ice matrix. Sand in which all the water is more or less frozen exhibits a brittle type of failure at low strains, for example, at around 2% strain. The water content of coarse soils is converted almost wholly into ice at a very few degrees below freezing point. Hence, frozen coarse soils exhibit a reasonably high compressive strength only a few degrees below freezing, and there is justification for using this parameter as a design index of their performance in the field, provided that a suitable factor of safety is incorporated. The order of increase in compressive strength with decreasing temperature is shown in Figure 5.14.

On the other hand, frozen clay, in addition to often containing a lower content of ice than sand, has layers of unfrozen water (of molecular proportions) around the clay particles. These molecular layers of water contribute towards a plastic type of failure. In fact, in fine sediments, the intimate bond between the water and clay particles results in a significant proportion of soil moisture remaining unfrozen at temperatures as low as –25°C. The more the clay material in the soil, the greater is the quantity of unfrozen moisture. As far as the unconfined compressive strength of frozen clays is concerned, there is a dramatic increase in strength with decreasing temperature. In fact, it appears to increase exponentially with the relative proportion of frozen moisture. Using silty clay as an example, the amount of moisture frozen at –18°C may be only 1.25 times that frozen at –5°C, but the increase in compressive strength may be more than fourfold.

Because frozen ground is more or less impermeable, this increases the problems due to thaw by impeding the removal of surface water. What is more, when thaw occurs, the amount of water liberated may greatly exceed that originally present in the melted out layer of the soil (see below). As the soil thaws downwards, the upper layers become saturated, and since

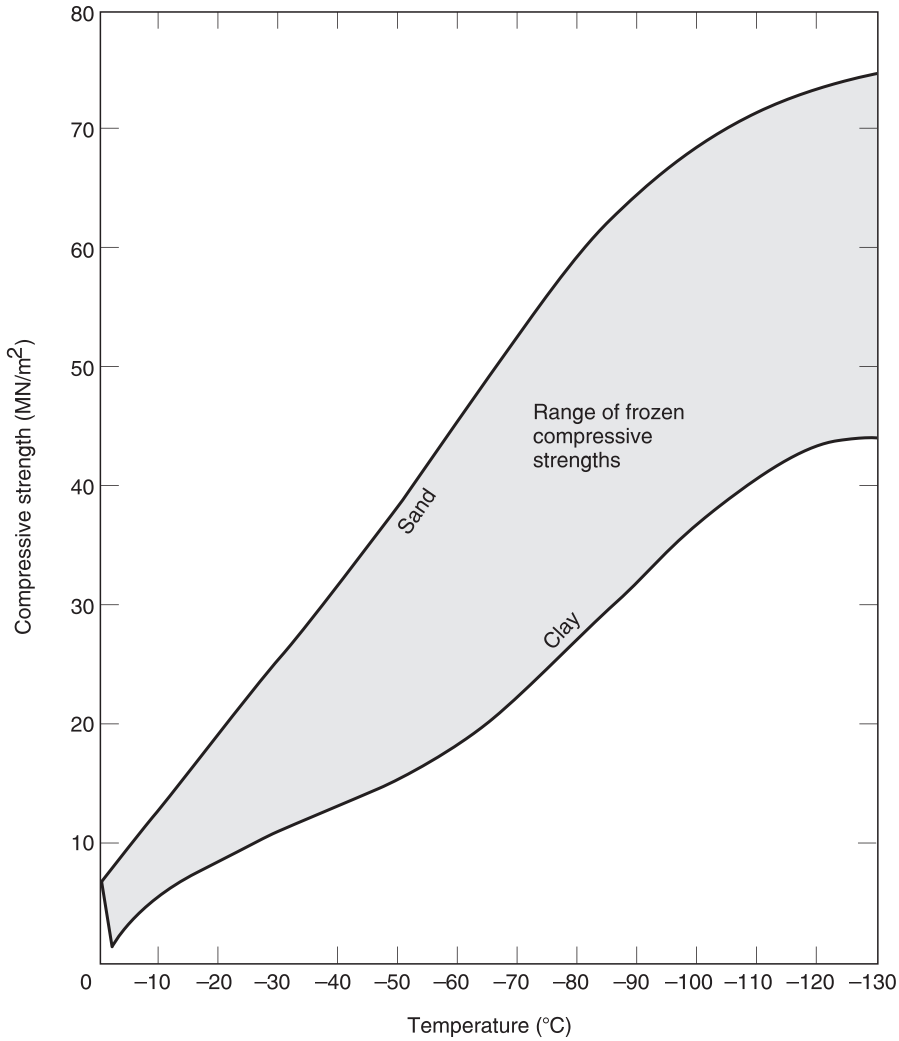


Figure 5.14

Increase in compressive strength with decreasing temperature.

water cannot drain through the frozen soil beneath, they may suffer a complete loss of strength. Indeed, under some circumstances excess water may act as a transporting agent, thereby giving rise to soil flows.

Thaw settlement is associated with the thawing of frozen ground. As ice melts, settlement occurs, water being squeezed from the ground by overburden pressure or by any applied loads. Excess pore water pressures develop when the rate of ice melting is greater than the discharge capacity of the soil. Since excess pore pressures can lead to the failure of slopes and foundations, both the rate and amount of thaw settlement should be determined. Pore water pressures also should be monitored.

Further consolidation, due to drainage, may occur on thawing. If the soil was previously in a relatively dense state, then the amount of consolidation is small. This situation only occurs in coarse frozen soils containing very little segregated ice. On the other hand, some degree of segregation of ice is always present in fine frozen soils. For example, lenses and veins of ice may be formed when silts have access to capillary water. Under such conditions, the moisture content of the frozen silts significantly exceeds the moisture content present in their unfrozen state. As a result, when such ice-rich soils thaw under drained conditions, they undergo large settlements under their own weight.

Frost Heave

The following factors are necessary for the occurrence of frost heave, namely, capillary saturation at the beginning and during the freezing of soil, plentiful supply of subsoil water and soil possessing fairly high capillarity together with moderate permeability. Grain size is another important factor influencing frost heave. For example, gravels, sands and clays are not particularly susceptible to heave, whereas silts definitely are. The reason for this is that silty soils are associated with high capillary rises, but at the same time their voids are large enough to allow moisture to move quickly enough for them to become saturated rapidly. If ice lenses are present in clean gravels or sands, then they simply represent small pockets of moisture that have been frozen. Casagrande (1932) suggested that the particle size critical to the development of frost heave was 0.02 mm. If the quantity of such particles in a soil is less than 1%, then no heave is to be expected, but considerable heaving may take place if the amount is over 3% in non-uniform soils and over 10% in very uniform soils.

As heave amounting to 30% of the thickness of the frozen layer have frequently been recorded, moisture other than that initially present in the frozen layer must be drawn from below, since water increases in volume by only 9% when frozen. In fact, when a soil freezes, there is an upward transfer of heat from the groundwater towards the area in which freezing is occurring. The thermal energy, in turn, initiates an upward migration of moisture within the soil. The moisture in the soil can be translocated upwards either in the vapour or liquid phase or by a combination of both. Moisture diffusion by the vapour phase occurs more readily in soils with larger void spaces than in fine soils. If a soil is saturated, migration in the vapour phase cannot take place.

Organic Soils: Peat

Peat is an accumulation of partially decomposed and disintegrated plant remains that have been fossilized under conditions of incomplete aeration and high water content (Hobbs, 1986). Physico-chemical and biochemical processes cause this organic material to remain in a state of preservation over a long period of time.

Macroscopically, peaty material can be divided into three basic groups, namely, amorphous granular, coarse fibrous and fine fibrous peat (Landva and Pheeney, 1980). The amorphous granular peat has a high colloidal fraction, holding most of its water in an adsorbed rather than free state. In the other two types, the peat is composed of fibres, these usually being woody. In the coarse variety, a mesh of second-order size exists within the interstices of the first-order network, and in fine fibrous peat, the interstices are very small and contain colloidal matter.

The ash percentage of peat consists of the mineral residue remaining after its ignition, which is expressed as a fraction of the total dry weight. Ash contents may be as low as 2% in some highly organic peat, or it may be as high as 50%. The mineral material is usually quartz sand and silt. In many deposits, the mineral content increases with depth. The mineral content influences the engineering properties of peat.

The void ratio peat ranges between 9, for dense amorphous granular peat, and 25, for fibrous types with a high content of sphagnum (Table 5.21). It usually tends to decrease with depth within a peat deposit. Such high void ratios give rise to phenomenally high water contents. The latter is the most distinctive characteristic of peat. Indeed, most of the peculiarities in the physical characteristics of peat are attributable to the amount of moisture present. This varies according to the type of peat; it may be as low as 500% in some amorphous granular varieties, although values exceeding 3000% have been recorded from coarse fibrous varieties.

The volumetric shrinkage of peat increases up to a maximum and then remains constant, the volume being reduced almost to the point of complete dehydration. The amount of shrinkage that can occur generally ranges between 10 and 75% of the original volume of the peat, and it can involve reductions in void ratio from over 12 down to about 2.

Amorphous granular peat has a higher bulk density than the fibrous types. For instance, in the former, it can range up to 1.2 Mg m^{-3}, whereas in woody fibrous peat, it may be half this figure. However, the dry density is a more important engineering property of peat, influencing its behaviour under load. Dry densities of drained peat fall within the range of 65–120 kg m^{-3}. The dry density is influenced by the mineral content, and higher values than those quoted can be obtained

Table 5.21. Some properties of bog peat from Pant Dedwydd, North Wales (after Nichol and Farmer, 1998). With kind permission of Elsevier

Depth (m)	Moisture content (%)	pH	Organic content (%)	Bulk unit weight (kN m^{-3})	Dry unit weight (kN m^{-3})	Specific gravity	Initial void ratio (e_o)	Coefficient of volume change m_v (m^2 MN^{-1})		Compression index, C_c	
								a	*b*	*a*	*b*
1.5	894	4.0	86.2	10.4	1.05	1.51	13.38	11.34	2.17	7.02	10.76
2.0	561	4.4	67.6	10.2	1.55	1.67	9.77	8.91	1.46	4.13	5.42
2.5	620	3.8	69.0	11.6	1.61	1.65	9.24	11.12	2.23	4.91	7.87
3.0	795	3.8	75.7	11.2	1.26	1.59	11.61	11.74	2.11	6.38	9.17
3.5	971	4.3	61.5	10.1	0.94	1.73	17.40	11.77	1.94	9.33	12.31
4.0	662	4.1	81.6	10.2	1.34	1.54	10.49	10.26	1.60	5.08	6.34
4.5	583	4.1	63.8	10.3	1.51	1.70	10.26	11.51	1.66	5.58	6.44
5.5	943	4.4	79.9	10.6	1.02	1.56	14.29	10.80	1.57	7.17	8.28
6.5	965	4.3	75.6	9.9	0.92	1.59	16.28	8.76	2.32	6.14	13.82

Note: Load ranges, σ_v: (a) 12.5–25 kN m^{-2}; (b) 100–200 kN m^{-2}.

when peat possesses high mineral residues. The specific gravity of peat ranges from as low as 1.1 up to about 1.8, again being influenced by the content of mineral matter. Due to its extremely low submerged density, which may be between 15 and 35 kg m^{-3}, peat is prone to rotational failure or failure by spreading, particularly under the action of horizontal seepage forces.

In an undrained bog, the unconfined compressive strength is negligible, the peat possessing a consistency approximating to that of a liquid. The strength is increased by drainage to values between 20 and 30 kPa and the modulus of elasticity to between 100 and 140 kPa. When loaded, peat deposits undergo high deformations but the modulus of deformation tends to increase with increasing load. If peat is very fibrous, it appears to suffer indefinite deformation without planes of failure developing. On the other hand, failure planes nearly always form in dense amorphous granular peat.

If the organic content of a soil exceeds 20% by weight, consolidation becomes increasingly dominated by the behaviour of the organic material (Berry and Poskitt, 1972). For example, on loading, peat undergoes a decrease in permeability of several orders of magnitude. Moreover, residual pore water pressure affects primary consolidation, and considerable secondary consolidation further complicates settlement prediction.

Differential and excessive settlement is the principal problem confronting the engineer working on peaty soil (Berry et al., 1985). When a load is applied to peat, settlement occurs because of the low lateral resistance offered by the adjacent unloaded peat. Serious shearing stresses are induced even by moderate loads. Worse still, should the loads exceed a given minimum, then settlement may be accompanied by creep, lateral spreading, or in extreme cases, by rotational slip and upheaval of adjacent ground. At any given time, the total settlement in peat due to loading involves settlement with and without volume change. Settlement without volume change is more serious for it can give rise to the types of failure mentioned. What is more, it does not enhance the strength of peat.

Description of Rocks and Rock Masses

Description is the initial step in an engineering assessment of rocks and rock masses. Therefore, it should be both uniform and consistent in order to gain acceptance. The data collected regarding rocks and rock masses should be recorded on data sheets for subsequent processing. A data sheet for the description of rock masses and another for discontinuity surveys are shown in Figures 5.15 and 2.17, respectively.

The complete specification of a rock mass requires descriptive information on the nature and distribution in space of both the materials that constitute the mass (rock, water and air-filled voids)

Rock Mass Description Data Sheet

General information

Seq. No. | Site | Date (Day, Month, Year) | Operator | Method of location | Co-ordinates or chainage (metres) | Northings or chainage | Eastings | Elevation

Method of location:
1 By co-ordinates
2 Chainage
3 On attached map/ drawing/photograph

Locality type:
1 Natural exposure
2 Construction excavation
3 Trial pit
4 Trench
5 Adit
6 Tunnel

Size of locality:
1 > 10 m²
2 5–10 m²
3 1–5 m²
4 < 1 m²
5 Line survey

No. of supplementary sheets of discontinuity data | Sketch | Photograph | Field tests | Remarks

Sketch / Photograph:
0 No
1 Yes

Field tests: Specify type

Rock material information

Colour:
1 Light
2 Dark

1 pinkish
2 reddish
3 yellowish
4 brownish
5 olive
6 greenish
7 bluish
8 greyish

1 pink
2 red
3 yellow
4 brown
5 olive
6 green
7 blue
8 white
9 grey
0 black

Grain size:
1 Very coarse
2 Coarse (2–60 mm)
3 Medium (60 μm–2 mm)
4 Fine (2–60 μm)
5 Very fine (< 2 μm)

Compressive strength:
1 Very strong (> 100 MPa)
2 Strong (50–100 MPa)
3 Mod. strong (12.5–50 MPa)
4 Mod. weak (5–12.5 MPa)
5 Weak (1.25–5 MPa)
6 V. weak/hard (600–1250 kPa)
7 Very stiff (300–600 kPa)
8 Stiff (150–300 kPa)
9 Firm (80–150 kPa)
0 Soft (40–60 kPa)

Method of determining compressive strength:
1 Measured
2 Assessed

Rock type

Qualifying terms to describe rock

Rock mass information

Fabric:
1 Blocky
2 Tabular
3 Columnar

Block size:
1 Very large (> 8 m³)
2 Large (0.2–8 m³)
3 Medium (0.008–0.2 m³)
4 Small (0.0002–0.008 m³)
5 Very small (< 0.0002 m³)

State of weathering:
1 Fresh
2 Slightly
3 Moderately
4 Highly
5 Completely
6 Residual soil

No. of major discontinuity sets

Line surveys to determine discontinuity spacings

	Plunge of line	Trend of line	Length of line (metres)	No. of fractures	Spacing	Remarks
Line 1						
Line 2						
Line 3						

Discontinuity spacing:
1 Ext. wide (< 2 m)
2 Very wide (600 mm–2 m)
3 Wide (200–600 mm)
4 Mod. wide (60–200 mm)
5 Mod. narrow (20–60 mm)
6 Narrow (6–20 mm)
7 Very narrow (6 mm)

Figure 5.15

Rock mass data description sheet (after Anon, 1977). With kind permission of the Geological society.

and the discontinuities that divide it (Anon, 1977a). The intact rock may be considered a continuum or polycrystalline solid consisting of an aggregate of minerals or grains, whereas a rock mass may be looked upon as a discontinuum of rock material transected by discontinuities. The properties of the intact rock are governed by the physical properties of the materials of which it is composed and the manner in which they are bonded to each other. The parameters that may be used in a description of intact rock therefore include petrological name, mineral composition, colour, texture, minor lithological characteristics, degree of weathering or alteration, density, porosity, strength, hardness, intrinsic or primary permeability, seismic velocity and modulus of elasticity. Swelling and slake durability can be taken into account where appropriate, such as in the case of argillaceous rocks. The behaviour of a rock mass is, to a large extent, determined by the type, spacing, orientation and characteristics of the discontinuities present (see Chapter 2). As a consequence, the parameters that ought to be used in the description of a rock mass include the nature and geometry of the discontinuities, as well as its overall strength, deformation modulus, secondary permeability and seismic velocity. It is not necessary, however, to describe all the parameters for either an intact rock or a rock mass.

Intact rock may be described from a geological or engineering point of view. In the first case, the origin and mineral content of a rock are of prime importance, as is its texture and any change that has occurred since its formation. In this respect, the name of a rock provides an indication of its origin, mineralogical composition and texture (Table 5.22). Only a basic petrographical description of the rock is required when describing a rock mass.

The texture of a rock, in particular its grain size, exerts some influence on the physical properties of a rock, for example, finer-grained rocks are usually stronger than coarser-grained varieties (Table 5.23). The overall colour of a rock should be assessed by reference to a colour chart (e.g. the rock colour chart of the Geological Society of America). Rock material tends to deteriorate in quality as a result of weathering and/or alteration. Classifications based on the estimation and description of physical disintegration and chemical decomposition of originally sound rock are given in Chapter 3. Density and porosity are two fundamental properties of rocks. The density of a rock is defined as its mass per unit volume. It is influenced primarily by mineral composition on the one hand and the amount of void space on the other. As the proportion of void space increases, the density decreases. Anon (1979) grouped the dry density and porosity of rocks into five classes as shown in Table 5.24. The unconfined compressive strength of a rock may be regarded as the highest stress that a cylindrical specimen can carry when a unidirectional stress is applied, normally in an axial direction, to its ends. Although its application is limited, the unconfined compressive strength does allow comparisons to be made between rocks and affords some indication of rock behaviour under more complex stress systems (Tsiambaos and Sabatakakis, 2004). There are several scales of unconfined compressive strength; three are given in Table 5.25. Rocks have a much lower tensile than compressive strength. The ratio of

Table 5.22. Rock type classification (after Anon, 1979). With kind permission of Springer

Genetic/group			**Detrital sedimentary**					**Pyroclastic**	**Chemical/ organic**
Usual structure			Bedded						
Composition			Grains of rock,quartz, feldspar and clay minerals			At least 50% of grains are of carbonate		At least 50% of grains are of fine-grained igneous rock	
Grain size (mm)				Grains are of rock fragments			Limestone and Dolomite (undifferentiated)		
60	Very coarse-grained	Rudaceous	Boulders Cobbles	Rounded grains: conglomerate		Carbonate gravel	Calcirudite	Rounded grains: agglomerate	Saline rocks Halite Anhydrite
2	Coarse-grained		Gravel	Angular gains: breccia				Angular grains: volcanic breccia lapilli tuff	Gypsum
				Grains are mainly mineral fragments					Limestone
	Medium-grained	Arenaceous	Sand	Sandstone: Grains are mainly mineral fragments Quartz arenite: 95% quartz, voids empty or cemented Arkose: 75% quartz, up to 25% feldspar:voids empty or cemented Greywacke: 75% quartz, 15% fine detrital material: rock and feldspar fragments		Carbonate sand	Calcarenite	Volcanic ash Tuff	Dolomite Chert Flint
0.06	Fine-grained	Argillaceous	Silt	Siltstone: 50% fine-grained particles	Mudstone shale: fissile mudstone	Marlstone Carbonate silt	Calcisiltite chalk	Fine-grained tuff	
0.002	Very-fine grained		Clay	Claystone: 50% very fine-grained particles		Carbonate mud	Calcilutite	Very fine-grained tuff	Peat Lignite Coal
	Glassy amorphous								

Genetic/group		Metamorphic		Igneous			
Usual structure		Foliated		Massive			
Composition		Quartz, feldspars, micas, acicular dark minerals		Light-coloured minerals are quartz, feldspar, mica		Dark and light minerals	Dark minerals
				Acid rocks	Intermediate	Basic rocks	Ultrabasic
60	Very coarse-grained			Pegmatite			Pyroxenite and peridotite
2	Coarse-grained	Gneiss (ortho-, para-, Alternate layers of granular and flaky minerals	Marble Granulite	Granite	Diorite	Gabbro	Serpentinite
Grain size (mm)	Medium-grained	Migmatite Schist Phyllite	Quartzite Hornfels Amphibolite	Microgranite	Microdiorite	Dolerite	
0.06	Fine-grained			Rhyolite	Andesite	Basalt	
0.002	Very fine-grained	Slate Mylonite					
	Glassy amorphous			Obsidian and pitchstone Volcanic glasses		Tachylyte	

Table 5.23. Description of grain size

Term	Particle size	Equivalent soil grade
Very coarse-grained	Over 60 mm	Boulders and cobbles
Coarse-grained	2–60 mm	Gravel
Medium-grained	0.006–2 mm	Sand
Fine-grained	0.002–0.06 mm	Silt
Very fine-grained	Less than 0.002	Clay

compressive to tensile strength generally is between 15:1 and 25:1. Unfortunately, however, the determination of the direct tensile strength frequently has proved difficult since it is not easy to grip the specimen without introducing bending stresses. Hence, most values of tensile strength quoted have been obtained by indirect methods of testing. One of the most popular of these methods is the point load test. Franklin and Broch (1972) suggested the scale for the point load strength shown in Table 5.26. The Schmidt hammer frequently is used as a means of assessing rock hardness. Unfortunately, the Schmidt hammer is not a satisfactory method for the determination of the hardness of very soft or very hard rocks, but there is a reasonably good correlation between Schmidt hardness and unconfined compressive strength. Young's modulus (deformability) is the ratio of vertical stress on a rock specimen, tested in unconfined compression, to strain. As far as deformability is concerned, the five classes shown in Table 5.27 have been proposed (Anon, 1979).

The durability of rocks is referred to in Chapter 3, the description of discontinuities in Chapter 2 and permeability in Chapter 4.

Engineering Aspects of Igneous and Metamorphic Rocks

The plutonic igneous rocks are characterized by granular texture, massive structure and relatively homogeneous composition. In their unaltered state, they are essentially sound and durable with adequate strength for any engineering requirement (Table 5.28). In some

Table 5.24. Dry density and porosity (after Anon, 1979). With kind permission of Springer

Class	Dry density (Mg m^{-3})	Description	Porosity (%)	Description
1	Less than 1.8	Very low	Over 30	Very high
2	1.8–2.2	Low	30–15	High
3	2.2–2.55	Moderate	15–5	Medium
4	2.55–2.75	High	5–1	Low
5	Over 2.75	Very high	Less than 1	Very low

Table 5.25. Grades of unconfined compressive strength. With kind permission of Springer

Geological society, (Anon, 1977)		IAEG, (Anon, 1979)		ISRM, (Anon, 1981)	
Term	**Strength (MPa)**	**Term**	**Strength (MPa)**	**Term**	**Strength (MPa)**
Very weak	less than 1.25	Weak	Under 15	Very low	Under 6
Weak	1.25–5.00	Moderately Strong	15–50	Low	6–10
Moderately weak	5.00–12.50			Moderate	20–60
Moderately strong	12.50–50	Strong	50–120	High	60–200
Strong	50–100	Very strong	120–230	Very high	Over 200
Very strong	100–200	Extremely strong	Over 230		
Extremely strong	over 200				

instances, however, intrusive rocks may be highly altered by weathering or hydrothermal attack; furthermore, fissure zones are by no means uncommon. The rock mass may be fragmented along such zones; indeed, it may be reduced to sand-size material, or it may have undergone varying degrees of kaolinization. Generally, the weathered product of plutonic rocks has a large clay content although that of granitic rocks is sometimes porous with permeability comparable to that of medium-grained sand. As would be expected, the character of the weathering is influenced by the climatic conditions under which weathering occurs. For instance, the degree of leaching that occurs during the chemical reactions governs the type of residual minerals that form. If only small amounts of cations are removed from the system, then montmorillonite or illite may be formed. On the other hand, should extensive eluviation processes develop, then kaolinite and, finally, gibbsite are produced. For example, Haskins et al. (1998) found that the granite saprolite at Injaka Dam site, Mpumalanga Province, South Africa, had formed under intense chemical weathering in well-drained conditions. Gibbsite and goethite occurred at the top of the profile where the most intense chemical weathering had taken place. The strength of granite undergoes a notable reduction on weathering. Some deeply weathered granites in South Africa have been identified as being

Table 5.26. Point load strength classification (after Franklin and Broch, 1972)

	Point load strength index (MPa)	**Equivalent uniaxial compressive strength (MPa)**
Extremely high strength	Over 10	Over 160
Very high strength	3–10	50–160
High strength	1–3	15–60
Medium strength	0.3–1	5–16
Low strength	0.1–0.3	1.6–5
Very low strength	0.03–0.1	0.5–1.6
Extremely low strength	Less than 0.03	Less than 0.5

Table 5.27. Classification of deformability (after Anon, 1979). With kind prmission of Springer

Class	Deformability (MPa $\times 10^{-3}$)	Description
1	Less than 5	Very high
2	5–15	High
3	15–30	Moderate
4	30–60	Low
5	Over 60	Very low

prone to dispersivity. For example, aggressive dispersivity was noted by Haskins et al. (1998). The relatively low values of dry density of the saprolite concerned (1.23–1.86 Mg m^{-3}) meant that it had a high void ratio (0.54–1.02). Such high void ratios also render the saprolite potentially collapsible.

Generally speaking, the older volcanic deposits do not prove a problem in foundation engineering, ancient lavas having strengths frequently in excess of 200 MPa. But volcanic deposits of geologically recent age prove treacherous at times, particularly if they have to carry heavy loads. This is because they often represent markedly anisotropic sequences in

Table 5.28. Geomechanical properties of some igneous and metamorphic rocks

	Specific gravity	Unconfined compressive strength (MPa)	Point load strength (MPa)	Schmidt hammer hardness	Young's modulus (GPa)
Mount Sorrel Granite	2.68	176.4 (VS)[a]	11.3 (EHS)[b]	54	60.6 (VL)[c]
Eskdale Granite	2.65	198.3 (VS)	12.0 (EHS)	50	56.6 (L)
Dalbeattie Granite	2.67	147.8 (VS)	10.3 (EHS)	69	41.1 (L)
Markfieldite	2.68	185.2 (VS)	11.3 (EHS)	66	56.2 (L)
Granophyre (Cumbria)	2.65	204.7 (ES)	14.0 (EHS)	52	84.3 (VL)
Andesite (Somerset)	2.79	204.3 (ES)	14.8 (EHS)	67	77.0 (VL)
Basalt (Derbyshire	2.91	321.0 (ES)	16.9 (EHS)	61	93.6 (VL)
Slate* (North Wales)	2.67	96.4 (S)	7.9 (VHS)	42	31.2 (L)
Slate⁺ (North Wales)		72.3 (S)	4.2 (VHS)		
Schist* (Aberdeenshire)	2.66	82.7 (S)	7.2 (VHS)	3`1	35.5 (L)
Schist⁺		71.9 (S)	5.7 (VHS)		
Gneiss	2.66	162.0 (VS)	12.7 (EHS)	49	46.0 (L)
Hornfels (Cumbria)	2.68	303.1 (ES)	20.8 (EHS)	61	109.3 (VL)

Note: *Tested normal to cleavage or schistosity; ⁺tested parallel to cleavage or schistosity.
[a]Classification of strength according to Anon (1977a): ES = extremely strong, over 200 MPa; VS = very strong, 100–200 MPa; S = strong, 50–100 MPa.
[b]Classification of point load strength according to Franklin, J.A. and Broch, E. 1972: EHS = extremely high strength, over 10 MPa; VHS = very high strength, 3–10 MPa.
[c]Classification of deformability according to Anon (1979): VL = very low, over 60 GPa; L = low, 30–60 GPa.

which lavas, pyroclasts and mudflows are interbedded. In addition, weathering during periods of volcanic inactivity may have produced fossil soils, these being of much lower strength. The individual lava flows may be thin and transected by a polygonal pattern of cooling joints. They also may be vesicular or contain pipes, cavities or even tunnels (see Chapter 1).

Normally, the geomechanical properties of fresh basalts and dolerites are satisfactory for engineering purposes. The values of some geomechanical properties of basalts are illustrated in Table 5.29. Bell and Jermy (2000) examined a number of the properties of some dolerites from South Africa. They found that the dry density of these dolerites ranged from 2.72 to 2.99 Mg m^{-3}, with a mean value of 2.93 Mg m^{-3}. Such high dry densities were reflected in low or, more commonly, very low porosities (i.e. less than 1%). Of the specimens tested in unconfined compression, 35% were extremely strong and 46% were very strong, according to the strength classification of Anon (1977a). In fact, the unconfined compressive strength ranged from 31.4 to 368.4 MPa, the lower values being associated with moderately or slightly weathered dolerites. In addition, the grain size appeared to exert a significant influence on strength. For example, the maximum difference in strength between fine- and medium-grained dolerite was 178 MPa, whereas the minimum difference was 51 MPa, the fine-grained dolerite being the stronger one. The range of Young's modulus for all dolerites extended from 40.9 to 100.5 GPa.

Certain basalts and dolerites are susceptible to rapid weathering. This rapid breakdown phenomenon is referred to as slaking. Certain factors are responsible for causing and/or enhancing breakdown of basalts and dolerites. These include swelling and shrinking of smectitic clays upon hydration and dehydration, respectively, and swelling and shrinking of

Table 5.29. Values of some geomechanical properties of basalts

Location	**Dry density (Mg m^{-3})**	**Unconfined compressive stength (MPa)**			**Point load strength (MPa)**			**Young's modulus (GPa)**		
Lesotho*		Min	Max	Mean	Min	Max	Mean	Min	Max	Mean
HAB	2.64	40	190	90						33.3
MAB	2.72	87	202	112						39.7
NAB	2.77	75	234	123						38.0
Turkey[+]	2.68	86.3	136.4	120.1	7.7	10.0	8.5	44.0	81.6	66.1
Greece[$]		43.1	91.2	74.2	1.0	3.4	2.5			
USA[ϕ]		81	413	266			14.5	22.0	127.9	78

Note: HAB = highly amygdaloidal basalt; MAB = moderately amygdaloidal basalt; NAB = non-amygdaloidal basalt.
* From Bell and Haskins (1997).
[+] From Tugrul and Gürpinar (1997).
[$] From Aggistalis et al. (1996).
[ϕ] From Schultz (1995).

particular zeolites. Repeated hydration and dehydration results in mechanical disruption of small portions of rock close to an exposed surface, causing flaking and surface cracking. The process is self-perpetuating as the formation of these cracks allows access of water into the rock, causing an increase in the degree and rate of weathering. Deuteric alteration of primary minerals brought about by hot gases and fluids from a magmatic source migrating through rock has been claimed to be responsible for the formation of secondary clay minerals in basalts and dolerites. The primary rock-forming minerals that tend to undergo the most deuteric alteration are olivine, plagioclase, pyroxene and biotite, and additionally volcanic glass when present in the case of basalt. Haskins and Bell (1995) suggested that if basalt contains between 10 and 20% of secondary montmorillonite, then it has the potential to degrade. They also noted that the disintegration of some basalt took the form of crazing, that is, extensive microfracturing that develops on exposure to the atmosphere or moisture. These microfractures expand with time, causing the basalt to disintegrate into gravel-sized fragments.

Pyroclasts usually give rise to extremely variable ground conditions due to wide variations in strength, durability and permeability. Their behaviour very much depends on their degree of induration, for example, many agglomerates have enough strength to support heavy loads and also have a low permeability. By contrast, ashes are weak and often highly permeable. They also may be metastable and exhibit a significant decrease in their void ratio on saturation. Ashes are frequently prone to sliding. Montmorillonite is not an uncommon constituent in the weathered products of basic ashes.

Slates, phyllites and schists are characterized by textures that have a marked preferred orientation. This preferred alignment of platey minerals accounts for cleavage and schistosity that typify these metamorphic rocks and means that slate, in particular, is notably fissile. Obviously, such rocks are appreciably stronger across than along the lineation. For example, Donath (1964) demonstrated that cores cut in Martinsburg Slates at 90° to the cleavage possessed the highest strength, whereas cores cut at 30° exhibited the lowest. These are high-density rocks with correspondingly low values of porosity. Not only does cleavage and schistosity adversely affect the strength of metamorphic rocks, it also makes them more susceptible to decay. Generally speaking, however, slates, phyllites and schists weather slowly but the areas of regional metamorphism in which they occur have suffered extensive folding so that rocks may be fractured and deformed in places. Schists, slates and phyllites are variable in quality, some being excellent foundations for heavy structures, others, regardless of the degree of their deformation or weathering, are so poor as to be wholly undesirable. For instance, talc, chlorite and sericite schist are weak rocks containing planes of schistosity only a millimetre or so apart. Some schists become slippery on weathering and therefore fail under a moderately light load.

The engineering performance of gneiss usually is similar to that of granite. However, some gneisses are strongly foliated, which means that they possess a texture with a preferred orientation. Generally, this will not significantly affect their engineering behaviour. Jayawardena and Izawa (1994) discussed the weathering of various gneisses from Sri Lanka and indicated their significant loss of strength on weathering, from approximately 180 MPa unconfined compressive strength for fresh rocks, to 160 MPa for slightly weathered, 90 MPa for moderately weathered and 11.5 MPa for highly weathered rocks. It can be seen from these values that the strength reduction accelerates with the degree of weathering. Gneisses may be fissured in places, and this can mean trouble. For instance, it would appear that fissures in the gneiss under the heel of the Malpasset Dam, France, opened, allowing the slow build-up of water pressure, which eventually led to its failure.

Fresh, thermally metamorphosed rocks such as quartzite and hornfels are very strong and afford good ground conditions. Marble generally has the same advantages and disadvantages as other carbonate rocks.

Engineering Behaviour of Sedimentary Rocks

Sandstones

Sandstones may vary from thinly laminated micaceous types to very thickly bedded varieties. Moreover, they may be cross-bedded and are jointed. With the exception of shaley sandstone, sandstone is not subject to rapid surface deterioration on exposure.

Studies of sandstones have shown that their geomechanical properties vary widely. Such variation has been attributed to differences in some of the petrographical characteristics of sandstones, including grain size distribution, packing density, packing proximity, type of grain contact, length of grain contact, amount of void space, type and amount of cement/matrix material and mineral composition. For instance, Bell and Culshaw (1993) demonstrated that in the sandstones from the Sherwood Sandstone Group in Nottinghamshire, England, those with smaller mean grain size possessed higher strength. Conversely, the particle size measures had no influence on either compressive or indirect tensile strength of the sandstones of the Sneinton Formation, Nottinghamshire, England, referred to by Bell and Culshaw (1998). Bell (1978) and Shakoor and Bonelli (1991) drew the same conclusion for the sandstones they examined. Dobereiner and de Freitas (1986) concluded that weak sandstones generally were characterized by low packing density. Castro and Bell (1995) indicated that the variation in the unconfined compressive strength of the sandstones from the Clarens Formation, South Africa, appeared to be related to the nature of the grain packing and the type of grain contact. In nearly all cases, those samples that had unconfined compressive strengths in excess of 40 MPa could be described as densely packed. The amount of grain contact (i.e. the ratio of the length of contact a grain has with its neighbours to its own total length, measured in two dimensions and expressed as a percentage) was regarded as exerting a major influence on the strength and deformability of sandstones by Dyke and Dobereiner (1991). Conflicting results also have been reported, relating to the influence of mineral content on the geomechanical properties of sandstones. For instance, Bell and Lindsay (1999) found that the unconfined compressive strength increased as the quartz content increased in the sandstones of Natal Group, South Africa. In contrast, Bell (1978) found no significant relationship between quartz content and strength in the sandstones from Fell Sandstones of Northumberland, England. The cement content and textural interlocking of the quartz grains was considered more important in terms of strength than quartz content. Ulusay et al. (1994) agreed with Bell that textural interlocking of quartz grains was more important than quartz content in terms of the Kozln Sandstone, Turkey. Bell noted that as the amount of cement increased in the Fell Sandstone, its strength also increased. Indeed, if cement binds the grains together, then a stronger rock is produced than one in which a similar amount of detrital material performs the same function. However, the amount of cementing material is more important than the type of cement. For example, ancient quartz arenites, in which the voids are almost completely occupied with siliceous material are extremely strong with crushing strengths exceeding 240 MPa. By contrast, poorly cemented sandstones may possess crushing strengths of less than 3.5 MPa. According to Bell and Lindsay (1999), increases in silica cement, in the form of quartz overgrowths, led to increases in strength in the sandstones of Natal Group, South Africa.

The dry density and, especially, the porosity of sandstone are influenced by the amount of cement and/or matrix material occupying the pores. Usually, the density of sandstone tends to increase with increasing depth below the surface, the porosity decreasing.

The compressive strength of sandstone is influenced by its porosity; the higher the porosity, the lower the strength. Pore water also plays a very significant role as far as the compressive strength and deformation characteristics of sandstone are concerned (Jeng et al., 2004). As can be seen from Table 5.30, it can reduce the unconfined compressive strengths by 30–60%. The amount of saturation moisture content contained by the sample does not appear to be the most important factor bringing about reduction in strength. Hawkins and McConnell (1992) maintained that sandstones with abundant clay minerals or rock fragments showed significant strength loss on wetting. They suggested that the reduction in strength may be related to softening and possible expansion of the clay mineral content. They further noted that the reduction in the modulus of deformation on wetting was progressive, being initiated at low moisture content. Hawkins and McConnell found that values generally decrease sharply with increase in moisture content. The indirect tensile strength of sandstones, as determined by the point load test, tends to be less than one-fifteenth of their unconfined compressive strength. The modulus of deformation of sandstones tends to decline as the porosity increases, and increases as the unconfined compressive strength and tensile strength increase.

Table 5.30. Some geomechanical properties of sandstones

	Fell Sandstone, Northumberland, UK	Sherwood Sandstsone, Nottinghamshire, UK	Sneinton Sandstone, Nottinghamshire, UK	Natal Group Sandstone, South Africa	Clarens Sandstone, South Africa
Specific gravity					
Range	2.64–2.71	2.61–2.76	2.70–2.77	2.66–2.70	—
Mean	2.67	2.68	2.73	2.68	—
Dry density ($Mg\ m^{-3}$)					
Range	2.14–2.40	1.77–1.96	2.22–2.31	2.40–2.54	1.93–2.58
Mean	2.32	1.83	2.25	2.48	2.27
Effective porosity (%)					
Range	6.5–20.5	24.0–28.8	8.9–14.8	2.4–9.3	1.5–19.2
Mean	9.8	26.2	12.0	6.3	15.1
Dry unconfined compressive strength (MPa)					
Range	33.2–112.4	6.3–20.8	17.4–39.8	77–214	13.6–159
Mean	74.1	11.8	28.4	136.6	61.7
Saturated unconfined compressive strength (MPa)					
Range	19.1–97.2	2.3–12.1	10.7–25.6	—	—
Mean	52.8	7.0	18.2	—	—
Point load strength (MPa)					
Range	0.2–9.5	—	1.03–2.42	6–13	1.2–8.7
Mean	4.4	—	1.55	9.3	3.5
Brazilian strength (MPa)					
Range	2.1–9.5	0.41–1.82	1.76–3.68	6–20	—
Mean	6.5	0.80	2.5	14.9	—
Schmidt hardness					
Range	22–52	—	—	39–49	—
Mean	37.4	—	—	44.1	—
Young's modulus (GPa)					
Range	27.3–46.2	3.24–9.83	1.4–5.2	19.5–99.9	3.8–14.3
Mean	32.7	6.16	3.68	50.8	6.8

The degree of resistance of sandstone to weathering depends on its mineralogical composition, porosity, amount and type of cement and the presence of any planes of weakness such as lamination. Sandstones are composed chiefly of quartz grains that are highly resistant to weathering, but other minerals present in lesser amounts may be suspect, for example, feldspars may become kaolinized. Calcareous cements are vulnerable to attack by weak acids. The durability of weak sandstones, in particular, frequently is suspect. For instance, Yates (1992) referred to certain sandstones from the Sherwood Sandstone Group, Stokes-on-Trent, England, which often disaggregated on saturation. In fact, such sandstones had a saturated unconfined compressive strength, when determined, of less than 0.5 MPa. The clay mineral content of sandstone can affect its durability. Bell and Culshaw (1998), for example, found a significant relationship between clay fraction and slake durability in the sandstones from the Sneinton Formation. The clay fraction ranged from 4 to 18%. Although most of these sandstones had a high or very high slake durability index, it declined with increasing clay fraction.

Mudrocks

Mudrocks include those rocks that possess a modal grain size within the silt and/or clay grade. In other words, mudrocks contain more than 50% clastic grains of less than 60 μm size, and hence their geomechanical behaviour is dominated by the fine material. Therefore, they include siltstones, shales, mudstones and claystones. In terms of mineralogy, the clay minerals and quartz are quantitatively the most important, and the quartz–clay minerals ratio influences their geotechnical properties. For example, the liquid limit of clay shales increases with increasing clay mineral content, the amount of montmorillonite, if present, being especially important. Because of their frequent high clay mineral content, mudrocks may possess weak strength, and their durability may be suspect. Therefore, they can prove problematic as regards engineering.

Shales are characterized by their fissility. Consolidation with concomitant recrystallization on the one hand and the parallel orientation of platey minerals, notably micas, on the other give rise to the fissility of shales. An increasing content of siliceous or calcareous material gives less fissile shale, whereas carbonaceous or organic shales are exceptionally fissile. Moderate weathering increases the fissility of shale by partially removing the cementing agents along the laminations or by expansion due to the hydration of clay particles.

The natural moisture content of shales varies from less than 5%, increasing to as high as 35% for some clayey shales (Table 5.31). When the natural moisture content of shales exceeds 20%, they frequently are suspect as they tend to develop potentially high pore water pressures. Usually, the moisture content in the weathered zone is higher than in the unweathered shale beneath. Depending on the relative humidity, many shales slake almost immediately when exposed to air. Desiccation of shale following exposure leads to the creation of

Table 5.31. Some geomechanical properties of mudrocks

	Tow Law*	Kirkheaton*	Wrexham* (weathered)	Bearpaw Shale[+]	Bearpaw Shale[+] (weathered)	Pierre Shale[#]	Pierre Shale[#] (weathered)
Natural moisture content (%)	5.9–10.5 7.3	—	6.4–14.3	19–27 19	25–36 35	15–38 23	26–38 34
Bulk density (Mg m^{-3})	2.43–2.56 2.51	—	2.04–2.12	—	—	—	—
Dry density (Mg m^{-3})	2.2–2.4 2.34	2.58–2.63	1.84–1.92	—	—	—	—
Unconfined compressive strength (MPa)	25.7–45.4 35.5	34.4–69.9	—	1–28	0.5–1.0	0.8–2.6 1.4	—
Point load index (MPa)							
Diametral	0.9–0.37	0.11–0.54	—	—	—	—	—
Axial	1.22–2.67	1.16–13.23					
Shear strength	—						
c (kPa)		0–50	—	—	—	—	—
$\phi°$		38–47					
Liquid limit (%)	—	—	31–41	50–150	80–150 120	55–204 122	— 133
Plasticity index (%)	—	—	15–16	30–130 95	30–130 95	35–175 95	— 103

*From Coal Measures in Britain (after Bell et al., 1997).
[+]Clay shales from Canada (after Hsu and Nelson, 1993).
[#]Clay shales from USA (after Hsu and Nelson, 1993).

negative pore pressures and consequent tensile failure of weak intercrystalline bonds. This leads to the production of shale particles of coarse sand to fine gravel size. Alternate wetting and drying causes a rapid breakdown of compaction shales. Low-grade compaction shales, in particular, undergo complete disintegration after several cycles of drying and wetting. Mudstones tend to break down along irregular fracture patterns, which, when well developed, can mean that these rocks disintegrate within one or two cycles of wetting and drying.

The swelling properties of certain clay shales have proven extremely detrimental to the integrity of many civil engineering structures (Fig. 5.16). Swelling is attributable to the absorption of free water by certain clay minerals, notably, montmorillonite, in the clay fraction of shale. Highly fissured overconsolidated shales have greater swelling tendencies than poorly fissured clayey shales, the fissures providing access for water.

The degree of packing, and hence the density and porosity of mudrocks, depend on their mineral composition and grain size distribution, their mode of sedimentation, their subsequent depth of burial and tectonic history, and the effects of diagenesis. The bulk and dry densities of some British cemented shales are given in Table 5.31. The effect of weathering on density also can be seen from Table 5.31. The porosity of shale may range from slightly under 5% to just over 50%. Argillaceous materials are capable of undergoing appreciable suction before pore water is removed, drainage commencing when the necessary air-entry suction is achieved (about pF 2). Under increasing suction pressure, the incoming air drives out water from mudrock and some shrinkage takes place in the fabric before air can offer support. Generally, as the natural moisture content increases, so the effectiveness of soil suction declines.

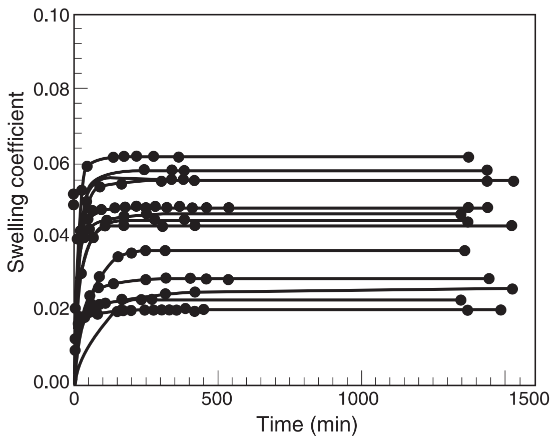


Figure 5.16

Free swell of carbonaceous mudrock (after Jermy and Bell, 1991).

When a load is applied to an essentially saturated shale foundation, the void ratio in the shale decreases and the pore water attempts to migrate to regions of lesser load. Because of its relative impermeability, water becomes trapped in the voids in the shale and can only migrate slowly. As the load is increased, there comes a point when it is in part transferred to the pore water, resulting in a build-up of pore water pressure. Depending on the permeability of the shale, and the rate of loading, the pore water pressure increases in value so that it equals the pressure imposed by the load. This greatly reduces the shear strength of the shale, and a serious failure can occur, especially in the weaker compaction shales. For instance, high pore water pressure in Pepper Shale was largely responsible for the foundation failure at Waco Dam, Texas (Underwood, 1967). Pore water pressure problems are not so important in cemented shales.

It would appear that the strength of compacted shales decreases exponentially with increasing void ratio and moisture content. In cemented shales, the amount and strength of the cementing material are the important factors influencing intact strength. The unconfined compressive strengths of some cemented shales from the Coal Measures of Britain and of compacted shales from North America are given in Table 5.31. Unconfined compressive strength tests on Accra Shales carried out by De Graft-Johnson et al. (1973) indicated that the samples usually failed at strains between 1.5 and 3.5%. The compressive strengths varied from 200 kPa to 20 MPa. Those samples that exhibited the high strengths were cemented types. Cemented shales not only are stronger but they are more durable than compacted shales. In weak compaction shales, cohesion may be lower than 15 kPa and the angle of friction as low as 5°. In contrast, Underwood (1967) quoted values of cohesion and angle of friction of 750 kPa and 56°, respectively, for dolomitic shales of Ordovician age, and 8–23 MPa and 45–64°, respectively, for calcareous and quartzose shales from the Cambrian period. Generally, shales with a cohesion of less than 20 kPa and angle of friction of less than 20° are likely to present problems. The elastic moduli of compaction shales range between 140 and 1400 MPa, whereas well-cemented shales have elastic moduli in excess of 14,000 MPa.

The higher the degree of fissility possessed by shale, the greater is the anisotropy with regard to strength, deformation and permeability. For instance, Wichter (1979) noted that in triaxial testing, the strength parallel to the laminations was some 1.5–2 times less than that obtained at right angles to it, for confining pressures of up to 1 MPa. Values of Young's modulus can be up to five times greater when shale is tested normal as opposed to parallel to the direction of lamination.

Severe settlements may take place in low-grade compaction shales. Conversely, uplift frequently occurs in excavations in shales and is attributable to swelling and heave. Rebound on unloading of shales during excavation is attributed to heave due to the release of stored strain energy. The greatest amount of rebound occurs in heavily overconsolidated compaction shales. Sulphur compounds frequently are present in shales and mudstones. An expansion in volume large enough to cause structural damage can occur when sulphide minerals such as pyrite and marcasite suffer oxidation to give anhydrous sulphates.

Clay shales usually have permeabilities of the order 1×10^{-8} m s^{-1}–10^{-12} m s^{-1}. However, sandy and silty shales and closely jointed cemented shales may have permeabilities as high as 1×10^{-6} m s^{-1}.

The greatest variation found in the engineering properties of mudrocks can be attributed to the effects of weathering. Weathering ultimately returns mudrock to a normally consolidated remoulded condition by destroying the bonds between particles. The lithological factors that govern the durability of mudrocks include the degree of induration, the degree of fracturing and lamination present, the grain size distribution and the mineralogical composition, especially the nature of the clay mineral fraction. On exposure and weathering at the surface, the laminae tend to separate, producing a fissile material that breaks down readily. Water can enter the rock more easily, and successive wetting and drying leads to its fragmentation. Weathering may take place preferentially along bedding planes in shales. For example, this has occurred along some bedding planes in the shale of the Pietermaritzburg Formation in Durban, South Africa (Bell and Lindsay, 1998). According to Bell and Maud (1996), this has given rise to the development of thin layers of clay, often 3–5 mm in thickness. The much reduced shear strength of these clay layers means that they represent preferential potential failure planes along which sliding can occur in the shale. If mudrock undergoes desiccation, then air is drawn into the outer pores and capillaries as high suction pressures develop. Then, on saturation, entrapped air is pressurized as water is drawn into the rock by capillarity. Therefore, such slaking causes the fabric of the rock to be stressed. Taylor (1988) maintained that disintegration takes place as a consequence of air breakage after a sufficient number of cycles of wetting and drying.

Carbonates

The engineering properties of carbonate sediments are influenced by grain size and those post-depositional changes that bring about induration. Because induration can take place at the same time as deposition is occurring, this means that carbonate sediments can sustain high overburden pressures that, in turn, means that they can retain high porosities to considerable depths. Indeed, a layer of cemented grains may overlie one that is poorly cemented. Eventually, however, high overburden pressure, creep and recrystallization produces crystalline limestone with very low porosity. Limestone is perhaps more prone to pre- and post-consolidation changes than any other rock type. For example, after burial, limestone can be modified to such an extent that its original characteristics are obscured or even obliterated. The most profound changes in composition and texture are those that lead to replacement of calcite by dolomite (dolomitization and the formation of dolostone), silica, phosphate and so forth.

Furthermore, carbonate rocks are susceptible to dissolution, which commonly removes shell fragments. Many limestones exhibit evidence of increases in grain size and crystallinity with increasing age of deposit.

Representative values of some physical properties of carbonate rocks are listed in Table 5.32. It can be seen that generally the density of these rocks increases with age, whereas the porosity is reduced. Diagenetic processes mainly account for the lower porosities of the older limestones. The porosity has a highly significant influence on the unconfined compressive

Table 5.32. Some geomechanical properties of British carbonate rocks (after Bell, 1981)

	Chee Tor Limestone Carboniferous (Buxton, Derbyshire)	**Magnesium Limestone Permian (Whitwell, Yorkshire)**	**Lincolnshire Limestone Jurassic (Ancaster, Lincolnshire)**	**Great Oolite Jurassic (Corsham, Wiltshire)**
Dry density (Mg m^{-3})				
Range	2.55–2.61	2.46–2.58	2.09–2.38	1.91–2.21
Mean	2.58	2.51	2.27	1.98
Porosity (%)				
Range	2.4–3.60	8.5–12.0	11.1–19.9	13.8–23.7
Mean	2.9	10.4	14.1	17.7
Dry unconfined compressive strength (MPa)				
Range	65.2–170.9	34.6–69.6	17.6–38.7	8.9–20.1
Mean	106.2	54.6	28.4	15.6
Saturated unconfined compressive strength (MPa)				
Range	56.1–131.6	25.6–49.4	10.7–22.4	7.8–10.4
Mean	83.9	36.6	16.8	9.3
Point load strength (MPa)				
Range	1.9–3.5	2.2–3.1	1.2–2.9	0.6–1.2
Mean	2.8	2.7	1.9	0.9
Schmidt hammer hardness				
Range	43–51	26–41	14–30	10–28
Mean	45	35	21	18
Young's modulus (GPa)				
Range	53.9–79.7	22.3–53.0	14.1–35.0	9.7–27.8
Mean	68.9	41.3	19.5	16.1
Permeability ($\times 10^{-8}$ m s^{-1})				
Range	0.01–0.3	1.8–8.4	6.9–41.9	17.2–45.2
Mean	0.07	4.1	17.5	26.6

strength, that is, as the porosity increases, the strength declines. Fookes and Hawkins (1988) mentioned that most Silurian, Devonian and Carboniferous limestones in Britain had at least a compressive strength of over 50 MPa, whereas Jurassic and Cretaceous limestones were often moderately weak, having unconfined compressive strength of less than 12.5 MPa. From Table 5.32, it can be seen that Carboniferous Limestone is generally very strong. Conversely, the Great Oolite, Jurassic, is only just moderately strong. This table also indicates that the unconfined compressive strength of the four limestones is reduced by saturation. The least reduction, that is 21%, is undergone by the limestone of Carboniferous age, which is also the strongest. The reduction in strength of the limestones from the Magnesium Limestone (Permian), Lincolnshire Limestone (Jurassic) and Great Oolite (Jurassic) is, respectively, 35%, 40% and 42%. In other words, the strongest material underwent the least reduction on saturation, and there was a progressive increase in the average percentage reduction undergone after saturation as the dry strength of the limestones decreased, with the least strong material showing the greatest reduction in strength. Similarly, the oldest limestones tend to possess the highest values of Young's modulus. Bell (1981) indicated that limestone from the Chee Tor Limestone (Carboniferous) tended to behave as a brittle material, exhibiting elastic deformation almost to the point of rupture, whereas limestone from the Magnesian Limestone, Lincolnshire Limestone and Great Oolite underwent varying degrees of plastic–elastic–plastic deformation prior to failure.

When dolomitized, limestone undergoes an increase in porosity of a few percent and, therefore, tends to possess a lower compressive strength than limestone that has not been dolomitized. For example, the Great Limestone of the north of England has a compressive strength ranging from 110 to 210 MPa with an average porosity of 4%. When dolomitized, its average porosity is 7.5% with a compressive strength of between 70 and 165 MPa. In fact, both dolomitization and dedolomitization can give rise to increased porosity in rocks. Williams and McNamara (1992) indicated that this can be responsible for lower compressive strength. They quoted the mean unconfined compressive strength for fresh limestones, dolostones, patchy dolostones and dedolostones from Cliff Dam site, Donegal, Ireland. These values were, respectively, 73.06 MPa, 57.09 MPa, 33.88 MPa and 17.23 MPa.

An important effect of dissolution in limestone is enlargement of the pores that enhances water circulation, thereby encouraging further dissolution. This brings about an increase in stress within the remaining rock framework that reduces the strength of the rock mass and leads to increasing stress corrosion. On loading, the volume of the voids is reduced by fracture of the weakened cement between the particles and by the reorientation of the intact aggregations of rock that become separated by loss of bonding. Most of the resultant settlement takes place rapidly within a few days of the application of load.

Dissolution of limestone is a very slow process. Nevertheless, it may be accelerated by man-made changes in groundwater conditions or by a change in the character of the surface water that drains into limestone. For instance, James and Kirkpatrick (1980) wrote that if such dry discontinuous rocks are subjected to substantial hydraulic gradients, then they will undergo dissolution along the discontinuities, leading to rapidly accelerating seepage rates. Joints in limestone generally have been subjected to various degrees of dissolution so that some may gape. Sinkholes may develop where joints intersect, and these may lead to an integrated system of subterranean galleries and caverns. The latter are characteristic of thick massive limestones. The progressive opening of discontinuities by dissolution leads to an increase in mass permeability.

Generally, chalk is a remarkably pure limestone, containing over 95% calcium carbonate that can be divided into coarse and fine fractions. The coarse fraction, which may constitute 20–30%, falls within the 40–100 μm range. This contains material derived from the mechanical breakdown of large-shelled organisms and, to a lesser extent, from foraminifera. The fine fraction, which takes the form of calcite particles that may be less than 1 μm in size, is composed almost entirely of coccoliths and may form up to, and sometimes over, 80% of certain horizons.

Chalk varies in hardness. For example, hard-grounds are present throughout much of the chalk in England. These are horizons, which may be less than 1 m or up to 10 m in thickness, that have undergone significant contemporaneous diagenetic hardening and densification. On the other hand, the individual particles in soft chalk, such as that in southeast England, are bound together at their points of contact by thin films of calcite. Such chalk contains only minute amounts of cement. Early cementation prevented gravitational consolidation occurring in soft chalk and helped retain high values of porosity. This contrasts with the Chalk in Yorkshire, in which in excess of 50% of the voids are occupied by cement due to overburden pressure bringing about pressure solution and precipitation of calcium carbonate (Bell et al., 1999). The Chalk in Humberside and Lincolnshire has been strengthened and hardened similarly.

The dry density of chalk has a notable range, for example, low values have been recorded from the Upper Chalk of Kent (1.35–1.64 Mg m^{-3}), whereas those from the Middle Chalk of Norfolk and the Lower Chalk of Yorkshire frequently exceed 2.0 Mg m^{-3}. Distinction often is made between hard and soft chalks on the basis of dry density. For example, Lord et al. (1994) suggested four classes, namely, low density (<1.55 Mg m^{-3}), medium density (1.55–1.70 Mg m^{-3}), high density (1.70–1.95 Mg m^{-3}) and very high density (>1.95 Mg m^{-3}). Chalk may have been deposited with as much as 70–80% porosity. Approximately half this pore space was lost by dewatering during the first tens to hundreds of metres of burial. Later, diagenetic processes during consolidation and cementation may have reduced this to less than 5%, although the average porosity of chalk is between about 25 and 40%. Price et al. (1976) measured the median pore diameters of Middle and Upper Chalk from Yorkshire, and obtained values of 0.39 and 0.41 μm, respectively, whereas the corresponding values from southern England and East Anglia were

0.53 and 0.65 μm, respectively. Generally, larger pores were found in the Upper Chalk and in the southern area. In southern England the median pore diameter in the Lower Chalk was 0.22 μm, a feature attributed, in part, to a high marl content.

The permeability of chalk is governed by its discontinuity pattern rather than by intergranular flow. As can be seen from the values given in Table 5.33, chalk has a high porosity but when the values are compared with intergranular permeability the relationship is poor. The values of primary permeability obtained by Bell et al. (1999) are more or less the same as those found by Ineson (1962). Ineson quoted a range between 0.1×10^{-10} and 25×10^{-9} m s^{-1}. The values provided by Bell et al. (1999) are shown in Table 5.33. The reason for the low primary permeability is the small size of the pores and, more particularly, that of the interconnecting throat areas. Price (1987) showed from mercury porosimeter testing that the median throat diameter typically was less than 1 μm, and that throat diameters are smaller in chalks from the north of England than in those from the south. Due to the operation of capillary and molecular forces, drainage of the "larger" pores (according to Price, their median diameters are approximately 5 μm) via such throats will not occur unless a suction on the order of 30 m head of water (approximately 300 kPa) is applied. Since gravitational drainage represents a suction of about 10 m, chalk has a very high specific retention.

The unconfined compressive strength of chalk ranges from moderately weak (much of the Upper Chalk) to moderately strong (much of the Lower Chalk of Yorkshire and the Middle Chalk of Norfolk). However, the unconfined compressive strength of chalk undergoes a marked reduction when it is saturated (Bell et al., 1999). For instance, the Upper Chalk from Kent may suffer a loss on saturation amounting to approximately 70%. Chalk compresses elastically up to a critical pressure, the apparent preconsolidation pressure. Marked breakdown and substantial consolidation occurs at higher pressures. The coefficients of consolidation, c_v, and volume compressibility, m_v, are around 1135 m^2 a^{-1} and 0.019 m^2 MN^{-1}, respectively.

The Upper Chalk from Kent is particularly deformable, a typical value of Young's modulus being 5×10^3 MPa. In fact, it exhibits elastic–plastic deformation, with perhaps incipient creep prior to failure. The deformation properties of chalk in the field depend on its hardness, and the spacing, tightness and orientation of its discontinuities. The values of Young's modulus also are influenced by the amount of weathering the chalk has undergone (Ward et al., 1968).

Discontinuities are the fundamental factors governing the mass permeability of chalk. Chalk also is subject to dissolution along discontinuities. However, subterranean solution features generally tend not to develop in chalk since it is usually softer than limestone and, hence, so collapses as solution occurs. Nevertheless, solution pipes and swallow holes are present in chalk.

Table 5.33. Some physical properties of chalk from Yorkshire, Norfolk and Kent, England (after Bell et al., 1999)

	Yorkshire*			Norfolk*				Kent***
	Lower	Middle	Upper	Lower	Melbourn Rock	Middle	Upper	Upper
Dry density (Mg m^{-3})								
Maximum	2.13 (L)	2.30 (M)	2.23 (M)	2.17 (L)	2.23 (M)	1.81 (L)	1.70 (VL)	1.61 (VL)
Minimum	1.85 (L)	1.76 (VL)	1.77 (VL)	1.71 (VL)	2.04 (L)	1.62 (VL)	1.54 (VL)	1.35 (VL)
Mean	2.08 (L)	2.14 (L)	2.06 (L)	1.99 (L)	2.17 (L)	1.76 (VL)	1.61 (VL)	1.44 (VL)
Effective porosity (%)(Saturation method)								
Maximum	30.2 (VH)	35.0 (VH)	36.4 (VH)	34.4 (VH)	27.0 (H)	38.2 (VH)	43.2 (VH)	45.7 (VH)
Minimum	17.2 (H)	16.2 (H)	17.7 (H)	19.9 (H)	16.1 (H)	30.2 (VH)	34.3 (VH)	29.6 (H)
Mean	20.6 (H)	21.8 (H)	23.9 (H)	26.5 (H)	19.8 (H)	34.4 (VH)	39.9 (VH)	41.7 (VH)
Permeability ($\times 10^{-9}$ m s^{-1})								
Maximum	1.2					2.2		37.0
Minimum	0.3					0.5		13.9
Mean	0.9					1.4		27.7
Dry unconfined compressive strength (MPa)**								
Maximum	32.7 (MS)	36.4 (MS)	34.0 (MS)	30.5 (MS)	38,3 (MS)	25.1 (MS)	12.7 (MS)	6.2 (MW)
Minimum	19.1 (MS)	25.2 (MS)	18.1 (MS)	14.2 (MS)	22.1 (MS)	7.4 (MW)	6.9 (MW)	4.8 (W)
Mean	26.4 (MS)	30.7 (MS)	25.6 (MS)	21.0 (MS)	29.1 (MS)	13.0 (MS)	9.5 (MW)	5.5 (MW)
Saturated unconfined compressive strength (MPa)								
Maximum	16.2	20.4	15.9	13.7	17.5	10.3	5.1	2.2
Minimum	8.6	11.7	7.4	6.2	8.9	3.1	2.8	1.4
Mean	13.7	16.8	11.9	10.7	14.3	5.8	3.6	1.7

Continued

Table 5.33. Some physical properties of chalk from Yorkshire, Norfolk and Kent, England (after Bell et al., 1999)—Cont'd

	Yorkshire*			Norfolk*				Kent***
	Lower	**Middle**	**Upper**	**Lower**	**Melbourn Rock**	**Middle**	**Upper**	**Upper**
Reduction of strength on saturation (%)	53	47	55	53	54	57	60	69
Point load strength (MPa)***								
Maximum	1.8 (HS)	2.1 (HS)	2.0 (HS)	1,5 (HS)	2.4 (HS)	—	—	—
Minimum	0.3 (MS)	0.6 (MS)	0.2 (LS)	0.2 (LS)	0.4 (MS)	—	—	—
Mean	1.4 (MS)	1.7 (HS)	1.2 (HS)	0.8 (MS)	1.7 (HS)	—	—	—
Young's modulus (t_{50} GPa)								
Maximum	18.4	21.7	17.1	14.1	18.9	10.4	8.2	4.6
Minimum	7.5	9.1	7.4	6.8	7.3	5.0	4.1	4.2
Mean	12.7	15.2	11.7	8.7	13.5	8.4	6.7	4.4

Note: Dry density VL = very low, less than 1.8 Mg m^{-3}; L = 1.8–2.2 Mg m^{-3}; M = moderate, over 2.2. Mg m^{-3}; Porosity H = high, 15–30%; VH = very high, over 30% (Anon, 1979). Unconfined compressive strength: W = weak, 1.25–5 MPa; MW = moderately weak, 5–12.5 MPa; MS = moderately strong, 12.5 to 50 MPa (Anon, 1977). Point load strength: LS = low strength, 0.1–0.3 MPa; MS = moderate strength,, 0.3–1 MPa; HS = high strength, 1–3 MPa (Franklin and Broch, 1972).

*Yorkshire: Lower—*H. subglobosus* (? = *S. gracile*) zone near Speeton; Middle — *T. lata* zone, Thornwick Bay; Upper—*M. coranguinum* zone, Selwicks Bay.

**Norfolk: Lower—*S. varians* (? = *M. mantelli*) zone, Hunstanton; Melbourn Rock and Middle—*T. lata zone*, Hilllington; Upper *M. coranguinum* zone, Burnham Market.

***Kent: Upper—*M. coranguinum* zone, Northfleet.

Evaporites

The dry densities of gypsum and anhydrite are given in Table 5.34, as are porosity values. There is little difference between the dry densities of anhydrite, and the range of values for the samples of gypsum also is low. The values of dry density of anhydrite are high, while those of gypsum are moderate, according to Anon (1979). Anhydrite is a strong rock, and gypsum is moderately strong. It would appear, however, that the purity of gypsum does have some influence on strength, as at Kirby Thore, the A bed was stronger than the purer B bed, and at Hawton, the grade 3 gypsum, which contained the most impurity, possessed the highest strengths among the samples tested (Table 5.34). It has been suggested by Skinner (1959) that impurities in calcium sulphate rocks tend to reduce the crystal size and that the strength increases with decreasing crystal size. Subsequently, Papadopoulos et al. (1994) investigated the influence of crystal size on the geotechnical properties of gypsum from Crete. They chose fine-grained alabaster, which probably was of basin deposition formed as a result of evaporation; medium-grained gypsum of secondary diagenetic origin; and large selenite crystals of primary displacement precipitation set in a host sediment. The point load test and unconfined compression test showed that alabaster had the highest strength, that is, that the finest-grained material was the strongest. The medium-grained gypsum possessed the least strength, with the selenitic material being somewhat stronger. Bell (1994) assessed the tensile strength of samples of anhydrite and gypsum indirectly by means of the point load test. All samples of anhydrite had a very high strength, whereas those of gypsum varied from low to high. Again, it appeared that the purer varieties possessed the lower strengths. Plastic deformation generally occurs at an earlier stage during the loading process in gypsum than it does in anhydrite. The values of Young's modulus are generally significantly higher for anhydrite than for gypsum. Indeed, if the average values of the two rock types are compared, then the E values of anhydrite tested by Bell tended to be at least twice those of gypsum (Table 5.34). In other words, the deformability of anhydrite is either very low or low, whereas that of gypsum varies from low to high.

Samples of anhydrite and gypsum were subjected to incremental loading creep tests and gypsum to conventional creep tests by Bell (1994). In both rocks, the amount of creep undergone in the incremental loading tests usually increased with increasing levels of constant loading. When gypsum was subjected to conventional creep tests, it underwent instantaneous elastic strain and then primary creep at a reasonably rapid but decelerating rate. This was followed by secondary creep at a low or near-constant strain rate.

The unconfined compressive strength of rock salt (halite) is generally moderately weak (9–15 MPa). A similar indication of strength is given by the point load test. Bell (1981) found that when halite was saturated with paraffin, the unconfined compressive strength suffered an average reduction of over 40%. Turning to Young's modulus, the values (e.g. 2–6.5 GPa)

Table 5.34. Geomechanical properties of anhydrite and gypsum, from the north and midlands of England, range and average values (after Bell, 1994c)

	Anhydrite		Gypsum					
			Kirby Thore				Hawton	
	Sandwich	Newbiggin	Bed A	Bed B	Sherburn-in-Elmet	Grade 1	Grade 2	Grade 3
Dry density ($Mg\ m^{-3}$)	2.77–2.82 2.79	2.74–2.84 2.78	2.16–2.33 2.29	2.19–2.32 2.24	2.16–2.32 2.21	2.21–2.24 2.22	2.23–2.26 2.25	2.22–2.24 2.23
Effective porosity (%)	3.1–3.7 3.3	3.0–3.5 2.9	3.6–4.6 4	3.9–7.0 4.4	3.4–9.1 5.1	1.5–5.0 2.3	2.8–4.4 3.2	3.5–6.6 3.8
Permeability ($m\ s^{-1}$)	$0.4–3.0 \times 10^{-8}$ 1.6×10^{-8}	$1.2–3.6 \times 10^{-8}$ 3.0×10^{-8}	$3.1–8.0 \times 10^{-7}$ 6.2×10^{-7}	$6.2–8.6 \times 10^{-7}$ 7.4×10^{-7}	4.0×10^{-7}–12.6×10^{-7} 9.6×10^{-7}	$3.2–6.6 \times 10^{-8}$ 4.8×10^{-8}	$2.3–5.3 \times 10^{-7}$ 3.65×10^{-7}	$8.9–11.8 \times 10^{-7}$ 10.6×10^{-7}
Unconfined compressive strength (MPa)	77.9–126.8 102.9	66.1–120.8 97.5	28.1–42.4 34.8	16.3–36.6 24.6	19.0–40.8 27.5	12.2–28.0 18.2	14.9–24.3 21.6	14.0–34.9 24.1
Point load strength (MPa)	3.4–4.9 4.0	3.0–4.6 3.4	1.8–2.5 2.0	1.1–1.8 1.5	0.9–2.4 1.9	0.21–1.85 1.34	1.15–1.69 1.40	0.4–2.42 1.65
Schmidt hammer hardness	36–43 37	33–40 35	20–28 23	18–26 20	17–34 20	0–14 8	0–17 8	6–22 12
Young's modulus (GPa) (Et_{50})	57.0–86.4 78.7	48.8–83.0 69.4	18.1–46.8 35.3	13.2–27.6 23.3	15.6–36.0 24.8	14.2–19.3 16.6	14.9–21.1 19.5	16.3–24.7 21.4

suggest that the rock is either very highly or highly deformable. In rock salt, the yield strength may be as little as one-tenth the ultimate compressive strength. Creep may account for anything between 20 and 60% of the strain at failure when rock salt is subjected to incremental creep tests. Rock salt is the most prone to creep of the evaporitic rocks.

Gypsum is more readily soluble than limestone; 2100 mg l^{-1} can be dissolved in non-saline waters as compared with 400 mg l^{-1} of limestone. Sinkholes and caverns can, therefore, develop in thick beds of gypsum more rapidly than they can in limestone. Cavern collapse has led to extensive cracking and subsidence at the ground surface. The problem is accentuated by the fact that gypsum is weaker than limestone and, therefore, collapses more readily. Rahn and Davis (1996) referred to the presence of sinkholes and caves in gypsum in the Rapid City area, South Dakota. Their presence has led to subsidence problems and, in some instances, sinkholes have collapsed suddenly. Cooper (1995) also commented on subsidence hazards due to the dissolution of gypsum. He maintained that natural rates of gypsum dissolution in the Ripon area of North Yorkshire, England, agreed with rates determined in the laboratory and elsewhere in the field. Furthermore, he added that the dissolution rate of gypsum from the sides of phreatic caves could be as high as 0.5–1 m per year under favourable flowing-water conditions. Many of the collapses have occurred after flooding or periods of prolonged rain, making their appearance within minutes. Where beds of gypsum approach the surface, their presence can be traced by broad funnel-shaped sinkholes formed by the collapse of overlying mudstone into areas from which gypsum has been removed by solution. Karstic features also have been described by Yuzer (1982) from the Sivas area of Turkey. Massive anhydrite can be dissolved to produce uncontrollable runaway situations in which seepage flow rates increase in a rapidly accelerating manner (James and Kirkpatrick, 1980).

Heave is another problem associated with anhydrite. This takes place when anhydrite is hydrated to form gypsum. In so doing, there is a volume increase of between 30 and 58% that exerts pressures that have been variously estimated between 2 and 69 MPa. It is thought that no great length of time is required to bring about such hydration. When it occurs at shallow depths, it causes expansion but the process is gradual and usually is accompanied by the removal of gypsum in solution. At greater depths, anhydrite is effectively confined during the process. This results in a gradual build-up of pressure and finally the stress is liberated in a rapid manner.

Rock salt is even more soluble than gypsum, and the evidence of slumping, brecciation and collapse structures in rocks that overlie saliferous strata bear witness to the fact that rock salt has gone into solution in past geological times. It generally is believed, however, that in humid regions that are underlain by saliferous beds, measurable surface subsidence is unlikely to occur, except where salt is being extracted. Perhaps this is because equilibrium has been attained between the supply of unsaturated groundwater and the rock salt available for solution. Nonetheless, cases have been recorded of rapid subsidence, such as the “Meade salt sink” in Kansas. This area of water, about 60 m in diameter, formed as a result of rapid subsidence in March 1879. Neal (1995) mentioned that more than 300 sinkholes, fissures and depressions are associated with the salt karst zone of the arid area of northeastern Arizona. Neal also mentioned salt karst in other evaporite basins such as in New Mexico, west Texas and Kansas.

Geological Materials Used in Construction

Building or Dimension Stone

Stone has been used as a construction material for thousands of years. One of the reasons for this was its ready availability locally. Furthermore, stone requires little energy for extraction and processing. Indeed, stone is used more or less as it is found except for the seasoning, shaping and dressing that is necessary before it is used for building purposes.

A number of factors determine whether a rock will be worked as a building stone. These include the volume of material that can be quarried; the ease with which it can be quarried; the wastage consequent upon quarrying; and the cost of transportation; as well as its appearance and physical properties (Yavuz et al., 2005). As far as volume is concerned, the life of the quarry should be at least 20 years. The amount of overburden that has to be removed also affects the economics of quarrying. Obviously, there comes a point when removal of overburden makes operations uneconomic. However, at that point, stone may be mined if conditions are favourable. Weathered rock normally represents waste therefore the ratio of fresh to weathered rock is another factor of economic importance. The ease with which a rock can be quarried depends to a large extent on geological structures, notably the geometry of joints and bedding planes, where present. Ideally, rock for building stone should be massive, certainly it must be free from closely spaced joints or other discontinuities as these control block size. The stone should be free of fractures and other flaws. In the case of sedimentary rocks, where beds dip steeply, quarrying has to take place along the strike. Steeply dipping rocks can also give rise to problems of slope stability when excavated. On the other hand, if beds of rock dip gently, it is advantageous to develop the quarry floor along the bedding planes. The massive nature of igneous rocks such as granite means that a quarry can be developed in any direction, within the constraints of planning permission.

A uniform appearance is generally desirable in building stone. The appearance of a stone largely depends on its colour, which is determined by its mineral composition. Texture also affects the appearance of a stone, as does the way in which it weathers. For example, the weathering of some minerals, such as pyrite, may produce ghastly stains. Generally speaking, rocks of light colour are used as building stone.

The texture and porosity of a rock affect its ease of dressing, and the amount of expansion, freezing and dissolution it may undergo. For example, fine-grained rocks are more easily dressed than coarse varieties. The retention of water in a rock with small pores is greater than in one with large pores and so they are more prone to frost attack.

For usual building purposes, a compressive strength of 35 MPa is satisfactory, and the strength of most rocks used for building stone is well in excess of this figure (Table 6.1 and Table 6.2). In certain instances, tensile strength is important, for example, tensile stresses may be generated in a stone subjected to ground movements. However, the tensile strength of a rock, or more particularly its resistance to bending, is a fraction of its compressive strength. As far as building stone is concerned, hardness is a factor of small consequence, except where a stone is subjected to continual wear, such as in steps or pavings.

The durability of a stone is a measure of its ability to resist weathering and so to retain its original size, shape, strength and appearance over an extensive period of time. It is one of the most important factors that determines whether or not a rock will be worked for building stone (Sims, 1991). The amount of weathering undergone by a rock in field exposures or quarries affords some indication of its qualities of resistance. However, there is no guarantee that the durability is the same throughout a rock mass and, if it changes, it is far more difficult to detect, for example, than a change in colour.

According to Leary (1986), one of the tests that is frequently used in Britain to make an initial assessment of the durability of sandstone as a building material is the acid immersion test. This involves immersing specimens for 10 days in sulphuric acid of density 1.145 Mg m^{-3}. Stones that are unaffected by the test are regarded as being resistant to attack by acidic rain-water. Those stones that fail are not recommended for external use in polluted environments. A more severe test consists of immersing specimens in sulphuric acid with a density of 1.306 Mg m^{-3}. Experience has shown that this test is of particular value when the design life of a proposed building is exceptionally long. If a stone survives the acid immersion test intact, then it is subjected to a crystallization test. The crystallization test uses either magnesium or sodium sulphate. There are two types of test, namely, the severe test and the mild test. The former test uses a saturated solution that is very aggressive and is only recommended for use when the natural weathering conditions are particularly severe, or the stone is expected to have a particularly long life. The mild test uses a 15% solution. The specimens are subjected to 15 cycles of immersion and, after final washing to remove sulphate and drying, are weighed to determine the weight loss. The results are reported in terms of weight loss, expressed as a percentage of the initial dry weight, or as the number of cycles required to produce failure if a specimen is too fractured to be weighed before the fifteenth cycle has been completed. Conclusions relating to durability are obtained by comparing the results of the tests with the performance of stone of known weathering behaviour. This unfortunately is one of the shortcomings of the test since specific reference stones are seldom available. Leary used the two types of acid immersion test and two types of crystallization test to classify sandstones into six grades, as follows:

1. Class A sandstones pass a severe acid immersion test and a severe crystallization test.
2. Class B sandstones pass a mild acid immersion test and a severe crystallization test.
3. Class C sandstones pass a severe acid immersion test and a mild crystallization test.
4. Class D sandstones pass a severe acid immersion test but fail a mild crystallization test.
5. Class E sandstones pass a mild acid immersion test but fail a mild crystallization test.
6. Class F sandstones fail a mild acid immersion test and a mild crystallization test.

Damage can occur to stone by alternate wetting and drying. What is more, water in the pores of a stone of low tensile strength can expand enough when warmed to cause its disruption. For example, when the temperature of water is raised from 0 to 60°C, it expands some 1.5%, and this can exert a pressure of up to 52 MPa in the pores of a rock. Indeed, water can cause expansion within granite ranging from 0.004 to 0.009%, in marble from 0.001 to 0.0025% and in quartz arenites (sandstones) from 0.01 to 0.044%. The stresses imposed on masonry by expansion and contraction, brought about by changes in temperature and moisture content, can result in masonry between abutments spalling at the joints, blocks may even be shattered and fall out of place.

Frost damage is one of the major factors causing deterioration in a building stone (Ingham, 2005). Sometimes, small fragments are separated from the surface of a stone due to frost action but the major effect is gross fracture. Frost damage is most likely to occur on steps, copings, cills and cornices where rain and snow can collect. Damage to susceptible stone may be reduced if it is placed in a sheltered location. Most igneous rocks, and the better quality sandstones and limestones, are immune. As far as frost susceptibility is concerned, the porosity, tortuosity, pore size and degree of saturation all play an important role. As water turns to ice, it increases in volume, thus giving rise to an increase in pressure within the pores. This action is further enhanced by the displacement of pore water away from the developing ice front. Once ice has formed, the ice pressure rapidly increases with decreasing temperature, so that at approximately −22°C, ice can exert a pressure of 200 MPa (Winkler, 1973). Usually, coarse-grained rocks withstand freezing better than the fine-grained types. Indeed, the critical pore size for freeze–thaw durability appears to be about 0.005 mm. In other words, rocks with larger mean pore diameters allow outward

Table 6.1. Some properties of British sandstones used for building purposes (after Bell, 1992a)

Stone	Colour	Grain Size	Specific gravity	Dry density (Mg m^{-3})	Porosity (%)	Unconfined compressive strength (MPa)	Young's modulus (GPa)	Acid immersion test*	Crystallization test	Saturation coefficient	Durability classification
Hollington Trias Near Uttoxeter	Pink to red, mottled buff	Fine to medium grained	2.71	2.04	23.5	29	13.6	Passed	F9	0.71	D
Lazonby Permian Near Penrith	Dark pink to red	Fine to medium grained	2.68	2.38	9.3	40	21.8	Passed	37	0.47	B,C
Delph Coal Measures Wingerworth	Grey to deep buff	Fine to medium grained	2.68	2.33	13.5	62	36.8	Passed	33	0.63	B,C
Ladycross Coal Measures Near Hexham	Light grey to buff	Fine to medium grained	2.69	2.36	11.6	82	41.2	Passed	9	0.62	A
Birchover Namurian Near Matlock	Buff to pink	Medium to coarse grained	2.69	2.34	12.4	48	25.6	Passed	40	0.65	B,C

Stancliffe Namurian Near Matlock	Buff	Fine to medium grained	2.67	2.38	11.5	72	41.5	Passed	20	0.63	A,B
Blaxter Lr Carboniferous Elsdon	Buff	Fine to medium grained	2.67	2.24	16.6	50	35.4	Passed	56	0.59	B,C
Monmouth Old Red Sandstone	Red to pinkish brown	Fine to medium grained	2.69	2.43	8.8	22	17.4	Failed	_	0.59	E,F

Note: Leary (1986) used combinations of severe and mild acid immersion crystallization tests to classify sandstones into six grades of durability, A passed both severe tests whereas F failed both mild tests.

* The acid immersion test involves immersing specimens for 10 days in sulphuric acid. Stones that are unaffected are regarded as being resistant to attack by acidic rainwater. Those stones that fail are not recommended for external use in polluted environments.

****** The crystallization test uses either magnesium or sodium sulphate. The specimens are subjected to 15 cycles of immersion and, after final washing to remove sulphate and subsequent drying, are weighed to determine the weight loss. The results are reported in terms of weight loss, expressed as a percentage of the initial dry weight, or as the number of cycles required to produce failure if a specimen is too fractured to be weighed before the fifteenth cycle has been completed.

Table 6.2. Some properties of British limestones used for building purposes (after Bell, 1993)

Stone	Orton Scar	Anstone	Doulting	Ancaster	Bath	Portland	Purbeck
Age	Lr. Carboniferous	Magnesian Limestone	Inferior Oolite	Lincolnshire Limestone	Great Oolite	Portland	Purbeck
Location	Orton	Kiveton Park	Shepton Mallet	Ancaster	Monks Park	Isle of Portland	Swanage
Specific gravity	2.72	2.83	2.7	2.7	2.71	2.7	2.7
Dry density (Mg m^{-3})	2.59	2.51	2.34	2.27	2.3	2.25	2.21
Porosity (%)	4.4	10.4	12.8	19.3	18.3	22.4	9.6
Microporosity (% saturation)	54	23	30	60	77	43	62
Saturation coefficient	0.68	0.64	0.69	0.84	0.94	0.58	0.62
Unconfined compressive strength (MPa)	96.4	54.6	35.6	28.4	15.6	20.2	24.1
Young's modulus (GPa)	60.9	41.3	24.1	19.5	16.1	17.0	17.4
Velocity of sound (m s^{-1})	4800	3600	2900	2900	2800	3000	3700
Crystallization test (% wt loss)	1	5	8	20	52	13	3
Durability classification*	A	B	C	D	E	C	B

*A = excellent; E = performs best in sheltered positions in inland locations where pollution is low and frost activity infrequent; F = generally unsatisfactory.

drainage and escape of moisture from the frontal advance of the ice line and, therefore, are less frost susceptible. Fine-grained rocks that have over 5% sorbed water are often very susceptible to frost damage, whereas those containing less than 1% are very durable. Freezing tests have proved an unsatisfactory method of assessing frost resistance. Capillary tests have been used in France and Belgium to assess the frost susceptibility of building stone, whereas in Britain, a crystallization test has been used (Anon, 1983).

Deleterious salts, when present in a building stone, are generally derived from the ground or the atmosphere, although soluble salts may occur in the pores of the parental rock. Their presence in a stone gives rise to different effects. They may cause efflorescence by crystallizing on the surface of a stone. In subflorescence, crystallization takes place just below the surface and may be responsible for surface scabbing. The pressures produced by crystallization of salts in small pores are appreciable, for instance, halite (NaCl) exerts a pressure of 200 MPa; gypsum ($CaSO_4 \cdot nH_2O$), 100 MPa; anhydrite ($CaSO_4$), 120 MPa and kieserite ($MgSO_4 \cdot nH_2O$), 100 MPa; and are often sufficient to cause disruption. Crystallization caused by freely soluble salts such as sodium chloride, sodium sulphate or sodium hydroxide can lead to the surface of a stone crumbling or powdering. Deep cavities may be formed in magnesian limestone when it is attacked by magnesium sulphate (Fig. 6.1). Salt action can

Figure 6.1

A cavity formed in magnesian limestone, parish church, Retford, Nottinghamshire, England.

give rise to honeycomb weathering in some sandstones and porous limestones (Fig. 3.3). Disruption in stone also may take place due to the considerable contrasts in thermal expansion of salts in the pores. For instance, halite expands by some 0.5% from 0 to 60°C, and this may aid the decay of stone. Conversely, surface induration of a stone by the precipitation of salts may give rise to a protective hard crust, that is, case hardening. If the stone is the sole supplier of these salts, then the interior is correspondingly weaker.

The rate of weathering of silicate rocks is usually slow, although once weathering penetrates the rock, the rate accelerates. Even so, building stones that are cut from igneous rocks generally suffer negligible decay in climates such as that of Britain. By contrast, some basalts used in Germany have proved exceptional in this respect in that they have deteriorated rapidly, crumbling after about 5 years of exposure. On petrological examination, these basalts were found to contain analcite, the development within which of micro-cracks is presumed to have produced the deterioration. Such basalts have been referred to as sun-burnt basalts. Haskins and Bell (1995) commented on the rapid breakdown of some basalts on exposure and attributed this to the presence of smectitic clay minerals formed by the deuteric alteration of primary minerals, the clay minerals swelling and shrinking on wetting and drying, respectively (see Chapter 5). This type of basalt has been termed slaking basalt. Some igneous stones weather a different colour. For example, within several weeks, some light grey granites may alter to various shades of pink, red, brown or yellow. This is caused by the hydration of the iron oxides in them.

Building stones derived from sedimentary rocks may undergo a varying amount of decay in urban atmospheres, where weathering is accelerated due to the presence of aggressive impurities such as SO_2, SO_3, NO_3, Cl_2 and CO_2 in the air, which produce corrosive acids. Limestones are the most suspect. For instance, weak sulphuric acid reacts with the calcium carbonate of limestones to produce calcium sulphate. The latter often forms just below the surface of a stone and the expansion that takes place upon crystallization causes slight disruption. If this reaction continues, then the outer surface of the limestone begins to flake off. In the more sheltered parts of a building, such as under ledges or in protected areas of decoration, calcium sulphate remains in position to form hard black crusts (Butlin et al., 1985). Black crusts are a mixture of gypsum and soot particles. They have a dramatic effect on the appearance of buildings (Fig. 6.2). Scanning electron microscope studies have shown that black crusts have an open crystalline structure that permits penetration of moisture. This moisture can carry dissolved salts into the stone with further disruptive consequences.

The degree of resistance that sandstone offers to weathering depends on its mineralogical composition, texture, porosity, amount and type of cement/matrix, and the presence of any planes of weakness. Accordingly, the best type of sandstone for external use for building purposes is a quartz arenite that is well bonded with siliceous cement, has a low porosity and

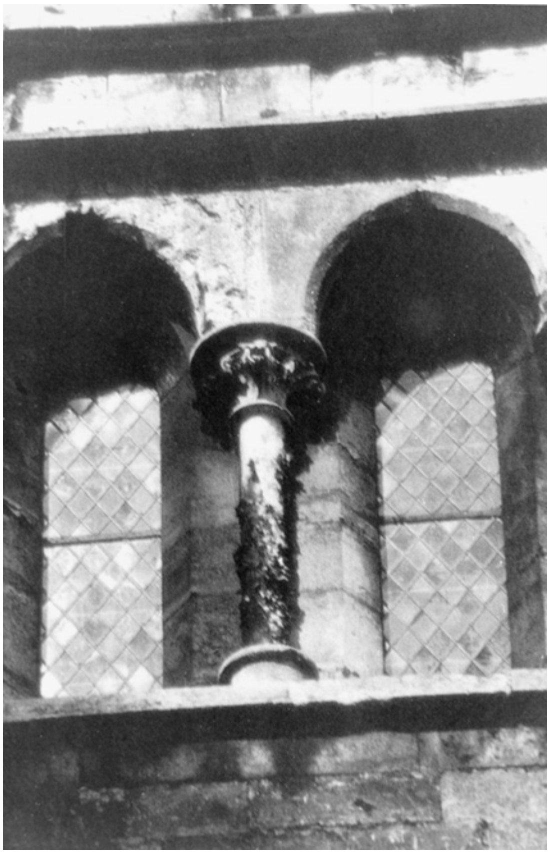


Figure 6.2

Black crust developed on a column of limestone, Lincoln cathedral, Lincoln, England.

is free from visible laminations. The tougher the stone, however, the more expensive it is to dress. Sandstones are chiefly composed of quartz grains that are highly resistant to weathering but other minerals present in lesser amounts may be suspect, for example, feldspars may be kaolinized. Calcareous cements react with weak acids in urban atmospheres, as do iron oxides that produce rusty surface stains. The reactions caused by acid attack may lead occasionally to the surface of a stone flaking off irregularly or, in extreme cases, to it crumbling. Laminated sandstone usually weathers badly when it is used in the exposed parts of buildings, it decaying in patches.

Exposure of a stone to intense heating causes expansion of its component minerals with subsequent exfoliation at its surface. The most suspect rocks in this respect appear to be those that contain high proportions of quartz and alkali feldspars, such as granites and sandstones. Indeed, quartz is one of the most expansive minerals, expanding by 3.76% between normal temperatures and 570°C. When limestones and marbles are heated to 900°C or dolostones to 800°C, superficial calcination begins to produce surface scars. Generally speaking, finely textured rocks offer a higher degree of heat resistance than do coarse-grained varieties.

Stone preservation involves the use of chemical treatments that prolong the life of a stone, either by preventing or retarding the progress of stone decay or by restoring the physical integrity of the decayed stone (Bell and Coulthard, 1990). A stone preservative, therefore, may be defined as a material that, when applied, will avert or compensate for the harmful effects of time and the environment. When applied, the preservative must not change the natural appearance or architectural value of the stone to any appreciable extent.

In some cases, stone may be obtained by splitting along the bedding and/or joint surfaces by using a wedge and feathers. Another method of quarrying rock for building stone consists of drilling a series of closely spaced holes (often with as little as 25 mm between them) in line in order to split a large block from the face. Stone also may be cut from the quarry face by using a wire saw or diamond-impregnated wire. Flame cutting has been used primarily for winning granitic rocks. It is claimed that this technique is the only way of cutting stone in areas of high stress relief. If explosives are used to work building stone, then the blast should only weaken the rock along joint and/or bedding planes, and not fracture the material. The object is to obtain blocks of large dimensions that can be sawn to size. Hence, the blasting pattern and amount of charge (black powder) are very important, and every effort should be made to keep rock wastage and hair cracking to a minimum.

When stone is won from a quarry, it contains a certain amount of pore water referred to as quarry sap. As this dries out, it causes the stone to harden. Consequently, it is wise to shape the stone as soon as possible after it has been got from the quarry. Blocks are first sawn to the required size, after which they may be planed or turned, before final finishing. Careless operation of dressing machines or tooling of the stone may produce bruising. Subsequently, scaling may develop at points where the stone was bruised, spoiling its appearance.

Granite is ideally suited for building, engineering and monumental purposes. Its crushing strength varies between 160 and 240 MPa. It has exceptional weathering properties, and most granites are virtually indestructible under normal climatic conditions. There are examples of granite polished over 100 years ago on which the polish has not deteriorated to any significant extent. Indeed, it is accepted that the polish on granite is such that it is only after exposure to very heavily polluted atmospheres, for a considerable length of time, that any sign of deterioration becomes apparent. The maintenance cost of granite as compared with other materials is therefore very much less and, in most cases, there is no maintenance cost at all for a considerable number of years.

Limestones show a variation in their colour, texture and porosity, and those that are fossiliferous are highly attractive when cut and polished. However, carbonate stone can undergo dissolution by acidified water. This results in dulling of polish, surface discolouration and structural weakening. Carvings and decoration are subdued and may eventually disappear; natural features such as grain, fossils, etc., are emboldened (Fig. 6.3).

The colour and strength of sandstone are largely attributable to the type and amount of cement binding the constituent grains. The cement content also influences the porosity and, therefore, water absorption. Sandstones that are used for building purposes are found in most of the geological systems, the exception being those of the Cainozoic era. The sandstones of this age are generally too soft and friable to be of value.

Roofing and Facing Materials

Rocks used for roofing purposes must possess a sufficient degree of fissility to allow them to split into thin slabs, in addition to being durable and impermeable. Consequently, slate is one of the best roofing materials available and has been used extensively. Today, however, more and more tiles are being used for roofing, these being cheaper than stone, which has to be quarried and cut to size.

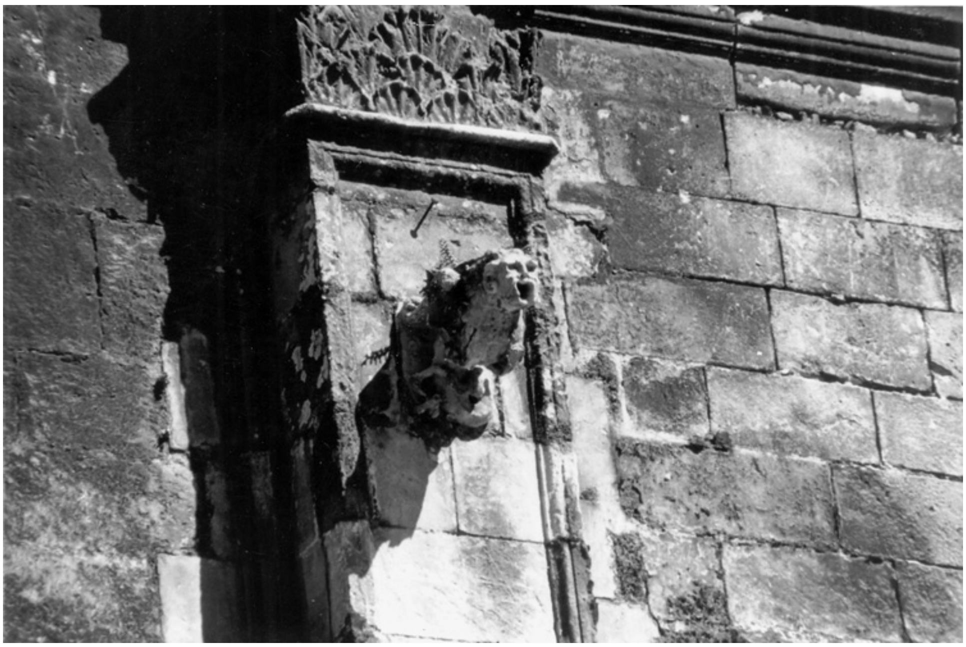


Figure 6.3

Weathered limestone gargoyle and scabbing of stone, Seville cathedral, Spain.

Slates are derived from argillaceous rocks that, because they were involved in major earth movements, were metamorphosed. They are characterized by their cleavage, which allows the rock to break into thin slabs. Some slates, however, may possess a grain that runs at an angle to the cleavage planes and may tend to fracture along it. Thus, in slate used for roofing purposes, the grain should run along its length. Welsh slates are differently coloured; they may be grey, blue, purple, red or mottled. The green coloured slates of the Lake District, England, are obtained from the Borrowdale Volcanic Series and are, in fact, cleaved tuffs. They are somewhat coarser grained than Welsh slates but more attractive. As noted earlier, the colour of slate varies. Red slates contain more than twice as much ferric as ferrous oxide. A slate may be greenish coloured if the reverse is the case. Manganese is responsible for the purplish colour of some slates. Blue and grey slates contain little ferric oxide.

The specific gravity of a slate is about 2.7 to 2.9, with an approximate density of 2.59 Mg m^{-3}. The maximum permissible water absorption of a slate is 0.37%. Calcium carbonate may be present in some slates of inferior quality that may result in them flaking and eventually crumbling upon weathering. Accordingly, a sulphuric acid test is used to test their quality. Top quality slates, which can be used under moderate to severe atmospheric pollution conditions, reveal no signs of flaking, lamination or swelling after the test.

There is a large amount of wastage when explosives are used to quarry slate. Accordingly, they are sometimes quarried by using a wire saw. The slate, once won, is sawn into blocks, and then into slabs about 75 mm thick. These slabs are split into slate tiles by hand. Riven facing stones are also produced in the same way (Fig. 6.4).

Today, an increasingly frequent method of using stone is as relatively thin slabs, applied as a facing to a building to enhance its appearance. Facing stone also provides a protective covering. Various thicknesses are used, from 20 mm in the case of granite, marble and slate in certain positions at ground-floor level, up to 40 mm at first-floor level or above. If granite or syenite is used as a facing stone, then it should not be overdried, but should retain some quarry sap, otherwise it becomes too tough and hard to fabricate. As far as limestones and sandstones are concerned, the slabs are somewhat thicker, that is, varying between 50 and 100 mm. Because of their comparative thinness, facing stones should not be too rigidly fixed; otherwise, differential expansion, due to changing temperatures, can produce cracking (Smith, 1999).

When fissile stones are used as facing stone and are given a riven or honed finish, they are extremely attractive. Facing stones usually have a polished finish, then they are even more attractive, the polished finish enhancing the textural features of the stone. Polishing is accomplished by carborundum-impregnated discs that rotate over the surface of the stone, successively finer discs being used to produce the final finish (Fig. 6.5). The discs are cooled

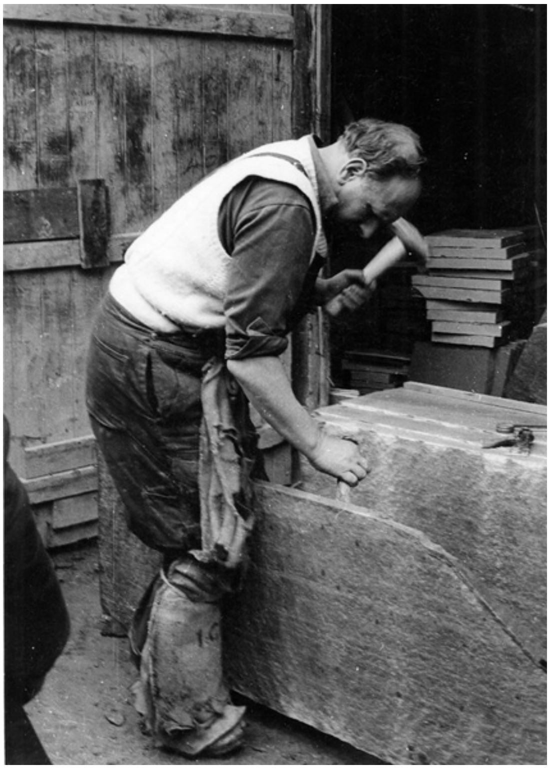


Figure 6.4

Coarse-grained “greenslate” being split by hand for facing stone, producing a riven finish. Broughton Moor, Cumbria, England.

and lubricated by water. A flame-textured finish can be produced by moving a small, high-temperature flame across the flat surface of a stone, which causes the surface to spall, thereby giving a rippled effect. Facing stones are almost self-cleansing.

Rocks used for facing stones should have a high tensile strength in order to resist cracking. The high tensile strength also means that thermal expansion is not a great problem when slabs are spread over large faces.

Armourstone

Armourstone refers to large blocks of rock that are used to protect civil engineering structures. Large blocks of rock, which may be single-size or, more frequently, widely graded (rip-rap), are used to protect the upstream face of dams against wave action. They are also used in the construction of river training schemes, in river bank and bed protection

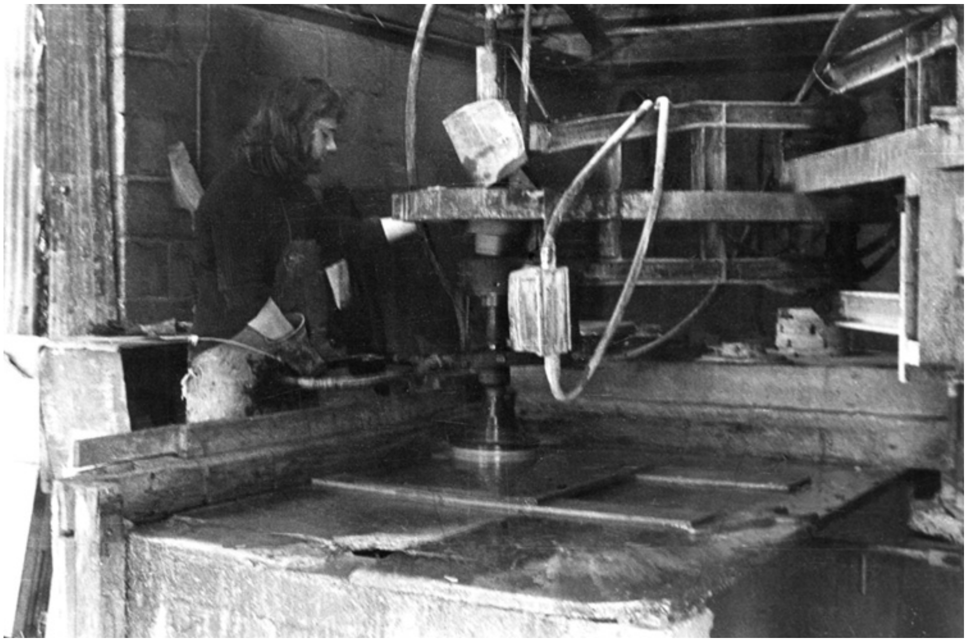


Figure 6.5

Carboniferous limestone being polished for facing stone, Orton, Cumbria, England.

and stabilization, as well as in the prevention of scour around bridge piers. Armourstone is used in coastal engineering for the construction of rubble mound breakwaters, for revetment covering embankments, for the protection of sea walls, and for rubble rock groynes. Indeed, breakwaters and sea defences represent a major use of armourstone. As the marine environment is one of the most aggressive in which construction occurs, armourstone must afford stability against wave action, accordingly block size and density are all important (Fookes and Poole, 1981). Shape is also important since this affects how blocks interlock together. In addition, armourstone must be able to withstand rapid and severe changes in hydraulic pressure, alternate wetting and drying, thermal changes, wave and sand/gravel impact and abrasion, as well as salt and solution damage. Consequently, the size, grading, shape, density, water absorption, abrasion resistance, impact resistance, strength and durability of the rock material used for armourstone must be considered during the design stage of a particular project. Hence, the selection of a suitable source of rock for armourstone requires an inspection and evaluation of the quarry or quarries concerned, as well as an assessment of the quality of the intact and processed stone (Latham, 1998). Moreover, the thickness of the protective layer and the need for a granular filter or geotextile beneath it depends on the design application, as well as the geotechnical and hydraulic conditions at the site (Thorne et al., 1995).

Usually, armourstone is specified by weight, a median weight of between 1 and 10 tonnes normally being required (Latham et al., 2006). Blocks up to 20 tonnes, however, may be required for breakwaters that will be subjected to large waves. The median weight of secondary armourstone and underlayer rock material may range upwards from 0.1 tonne. In the case of rip-rap used for revetment and river bank protection, the weight of the blocks required is usually less than 1.0 tonne and may grade down to 0.05 tonne. The size of blocks that can be produced at a quarry depends on the incidence of discontinuities and, to a lesser extent, on the method of extraction. Detailed discontinuity surveys can provide the data required for prediction of in situ block size and shape.

The location of armourstone on a breakwater is an important factor that should be considered when making an assessment of rock durability. Rock durability concerns its resistance to chemical decay and mechanical disintegration, including reduction in size and change of shape, during its working life. The intrinsic properties of a rock such as mineralogy, fabric, grain size, grain interlock, porosity, and in the case of sedimentary rocks, the type and amount of cementation, all affect its resistance to breakdown. In addition, the amount of damage that armourstone undergoes is influenced by the action to which it is submitted. For instance, the damage suffered by armourstone used on breakwaters depends on the type of waves (plunging or breaking), their height, period and duration (notably during storm conditions) on the one hand, and the slope and permeability of the structure on the other. Abrasion due to wave action is the principal reason for the reduction in size of blocks of armourstone used in breakwaters, as well as in rounding their shape.

Crushed Rock: Concrete Aggregate

Crushed rock is produced for a number of purposes, the chief of which are for concrete and road aggregate. Approximately 75% of the volume of concrete consists of aggregate, therefore its properties have a significant influence on the engineering behaviour of concrete. Aggregate is divided into coarse and fine types, the former usually consisting of rock material that is less than 40 mm and larger than 4 mm in size. The latter is obviously less than 4 mm. Fine types less than 75 μm should not exceed 10% by weight of the aggregate.

The amount of overburden that has to be removed is an important factor in quarrying operations, for if this increases and is not useable, then a time comes when quarrying operations become uneconomic. The removal of weak overburden is usually undertaken by scrapers and bulldozers, the material being disposed of in spoil dumps on site. Unfortunately, in the case of weathered overburden, weathered profiles are frequently not a simple function of depth below the surface and can be highly variable. Furthermore, in humid tropical areas, in particular, weathered horizons may extend to appreciable depths. Consequently, assessment of the amount of overburden that has to be removed can be complicated. Indurated overburden may require drilling and blasting before being removed by dump trucks to the dump area. Spoil is normally used to backfill worked out areas as part of the restoration programme.

High explosives such as gelegnite, dynamite or trimonite are used in drillholes when quarrying crushed rocks. ANFO, a mixture of diesel oil and ammonium nitrate, is also used frequently. The holes are drilled at an angle of about 10–20° from vertical for safety reasons, and are usually located 3–6 m from the working face and a similar distance apart. Generally one, but sometimes two, rows of holes are drilled. The explosive does not occupy the whole length of a drillhole, lengths of explosive alternating with zones of stemming, which is commonly quarry dust or sand. Stemming occupies the top six or so metres of a drillhole. A single detonation fires a cordex instantaneous fuse, which has been fed into each hole. It is common practice to have millisecond delay intervals between firing individual holes, in this way the explosions are complementary. The object of blasting is to produce a stone of workable size. Large stones must be further reduced by using a drop-ball or by secondary blasting. The height of the face largely depends on the stability of the rock mass concerned. When the height of a working face begins to exceed 20–30 m, it may be worked in tiers (Fig. 6.6). After quarrying, the rock is fed into a crusher and then screened to separate the broken rock material into different grade sizes.

Figure 6.6

Quarrying granite for aggregate, Hong Kong.

The crushing strength of rock used for aggregate generally ranges between 70 and 300 MPa. Aggregates that are physically unsound lead to the deterioration of concrete, inducing cracking, popping or spalling. Cement shrinks on drying. If the aggregate is strong, the amount of shrinkage is minimized and the cement–aggregate bond is good.

Hammersley (1989) noted that the petrography of a rock mass, involving field inspection, can be of value in any assessment of its potential suitability for use as aggregate. In addition, petrographic examination can indicate the presence of deleterious materials and defects.

The shape of aggregate particles is an important property and is governed mainly by the fracture pattern within a rock mass. Rocks such as basalts, dolerites, andesites, granites, quartzites and limestones tend to produce angular fragments when crushed. However, argillaceous limestones, when crushed, produce an excessive amount of fines. The crushing characteristics of sandstone depend on the closeness of its texture, and the amount and type of cement. Angular fragments may produce a mix that is difficult to work, that is, it can be placed less easily and offers less resistance to segregation. Nevertheless, angular particles are said to produce a denser concrete. Rounded, smooth fragments produce workable mixes. The less workable the mix, the more sand, water and cement must be added to produce a satisfactory concrete. Fissile rocks such as those that are strongly cleaved, schistose, foliated or laminated have a tendency to split and, unless crushed to a fine size, give rise to tabular- or planar-shaped particles. Planar and tabular fragments not only make concrete more difficult to work, but they also pack poorly and so reduce its compressive strength and bulk weight. Furthermore, they tend to lie horizontally in the cement, allowing water to collect beneath them, which inhibits the development of a strong bond on their under surfaces.

The surface texture of aggregate particles largely determines the strength of the bond between the cement and themselves. A rough surface creates a good bond, whereas a smooth surface does not.

As concrete sets, hydration takes place, and alkalies (Na_2O and K_2O) are released. These react with siliceous material such as opal, chalcedony, flint, chert and volcanic glass. If any of these materials are used as aggregate in concrete made with high-alkali cement, then the concrete is liable to expand and crack, thereby losing strength. Expansion due to alkali aggregate reaction has also occurred when greywacke was used as aggregate. When concrete is wet, the alkalies that are released are dissolved by its water content and, as the water is used up during hydration, the alkalies are concentrated in the remaining liquid. This caustic solution attacks reactive aggregates to produce alkali–silica gels. The osmotic pressures developed by these gels as they absorb more water may eventually rupture the cement around reacting aggregate particles. The gels gradually occupy the cracks thereby produced, and they eventually extend to the surface of the concrete. If alkali reaction is severe, a polygonal pattern of cracking develops on the surface. These troubles can be avoided if a preliminary petrological examination is made of the aggregate. In other words, material that contains over 0.25% opal, over 5% chalcedony, or over 3% glass or cryptocrystalline acidic to intermediate volcanic rock, by weight, will be sufficient to produce an alkali reaction in concrete, unless low-alkali cement is used. This contains less than 0.6% of Na_2O and K_2O. If aggregate contains reactive material surrounded by or mixed with inert matter, a deleterious reaction may be avoided. The deleterious effect of alkali aggregate reaction can also be avoided if a pozzolan is added to the mix, the reaction taking place between it and the alkalis.

Reactivity may be related not just to composition but also to the percentage of strained quartz that a rock contains. For instance, Gogte (1973) maintained that rock aggregates containing 40% or more of strongly undulatory or highly granulated quartz were highly reactive, whereas those with between 30 and 35% were moderately reactive. He also showed that basaltic rocks with 5% or more secondary chalcedony or opal, or about 15% palagonite, showed deleterious reactions with high-alkali cements. Sandstones containing 5% or more chert behaved in a similar manner.

Certain argillaceous dolostones have been found to expand when used as aggregates in high-alkali cements, thereby causing failure in concrete. This phenomenon has been referred to as alkali–carbonate rock reaction, and its explanation has been attempted by Gillott and Swenson (1969). They proposed that the expansion of such argillaceous dolostones in high-alkali cements was due to the uptake of moisture by the clay minerals. This was made possible by dedolomitization that provided access for moisture. Moreover, they noted that expansion only occurred when the dolomite crystals were less than 75 microns.

It usually is assumed that shrinkage in concrete should not exceed 0.045%, this taking place in the cement. However, basalt, gabbro, dolerite, mudstone and greywacke have been shown to be shrinkable, that is, they have large wetting and drying movements of their own, so much so that they affect the total shrinkage of concrete. Clay and shale absorb water and are likely to expand if they are incorporated in concrete, and they shrink on drying, causing injury to the cement. Consequently, the proportion of clay material in a fine aggregate should not exceed 3%. Granite, limestone, quartzite and felsite are unaffected.

Road Aggregate

Aggregate constitutes the basic material for road construction and is quarried in the same way as aggregate for concrete. Because it forms the greater part of a road surface, aggregate has to bear the main stresses imposed by traffic, such as slow-crushing loads and rapid-impact loads, and has to resist wear. Therefore, the rock material used should be fresh and have high strength. In addition, the aggregate used in the wearing course should be able to resist the polishing action of traffic. The aggregate in blacktop should possess good adhesion properties with bituminous binders.

Aggregate used as road metal must, in addition to having high strength, have high resistance to impact and abrasion, polishing and skidding, and frost action. It must also be impermeable, chemically inert and possess a low coefficient of expansion. The principal tests carried out in order to assess the value of a roadstone are the aggregate crushing test, the aggregate impact test, the aggregate abrasion test and the test for the assessment of the polished stone value. Other tests of consequence are those for water absorption, specific gravity and density, and the aggregate shape tests (Anon, 1975a). Some typical values of roadstone properties of rocks are given in Table 6.3.

The properties of an aggregate are related to the texture and mineralogical composition of the rock from which it was derived. Most igneous and contact metamorphic rocks meet the requirements demanded of good roadstone. On the other hand, many rocks of regional metamorphic origin are either cleaved or schistose and are therefore unsuitable for roadstone. This is because they tend to produce flaky particles when crushed. Such particles do not achieve good interlock and, consequently, impair the development of dense mixtures for surface dressing. The amount and type of cement and/or matrix material that bind grains together in a sedimentary rock influence roadstone performance.

The way in which alteration develops can influence roadstone durability. Weathering may reduce the bonding strength between grains to such an extent that they are plucked out easily from the stone. Chemical alteration is not always detrimental to roadstone performance; indeed a small amount of alteration may improve the resistance of a rock to polishing

Table 6.3. Some representative values of the roadstone properties of some common aggregates

Rock type	Water absorption	Specific gravity	Aggregate crushing value	Aggregate impact value	Aggregate abrasion value	Polished stone value
Basalt	0.9	2.91	14	13	14	58
Dolerite	0.4	2.95	10	9	6	55
Granite	0.8	2.64	17	20	15	56
Micro-granite	0.5	2.65	12	14	13	57
Hornfels	0.5	2.81	13	11	4	59
Quartzite	1.8	2.63	20	18	15	63
Limestone	0.5	2.69	14	20	16	54
Greywacke	0.5	2.72	10	12	7	62

(see the following text). On the other hand, resistance to abrasion decreases progressively with increasing content of altered minerals, as does the crushing strength. The combined hardness of the minerals in a rock, together with any degree of fissility, as well as the texture of the rock, also influence its rate of abrasion. The crushing strength is related to porosity and grain size; the higher the porosity and the larger the grain size, the lower the crushing strength.

One of the most important parameters of road aggregate is the polished stone value, which influences skid resistance. A skid-resistant surface is one that is able to retain a high degree of roughness while in service. At low speeds, the influence of the roadstone is predominant, whereas at high speeds, the influence of surface tension on skidding mainly depends on aggregate grading and the aggregate–binder relationship. The rate of polish is initially proportional to the volume of the traffic. Straight stretches of road are less subject to polishing than bends, which may polish up to seven times more rapidly. Stones are polished when fine detrital powder is introduced between the tyre and surface. Investigations have shown that detrital powder on a road surface tends to be coarser during wet than dry periods. This suggests that polishing is more significant when the road surface is dry than wet, the coarser detritus tending to roughen the surface of stone chippings. An improvement in skid resistance can be brought about by blending aggregates. The skid resistance value of the blend depends on the proportions of the individual materials composing the mix. Once placed in a road surface, however, the proportions of each component in the blend that are exposed influence the performance.

Rocks within the same major petrological group may differ appreciably in their polished stone characteristics. In the case of igneous and contact metamorphic rocks, the principal petrographic feature associated with good resistance to polish is a variation in hardness between the minerals present. In fact, the best resistance to polish occurs in rocks containing a proportion of softer alteration materials. Coarser grain size and the presence of cracks in individual grains also tend to improve resistance to polishing. In the case of sedimentary rocks, the presence of hard grains set in a softer matrix produces a good resistance to polish. Sandstones, greywackes and gritty limestones offer good resistance to polishing, but unfortunately not all of them possess sufficient resistance to crushing and abrasion to render them useful in the wearing course of a road. Purer limestones show a significant tendency to polish.

The petrology of an aggregate determines the nature of the surfaces to be coated, the adhesion attainable depending on the affinity between the individual minerals and the binder, as well as the surface texture of the individual aggregate particles. If the adhesion between the aggregate and binder is less than the cohesion of the binder, stripping may occur. Insufficient drying and the non-removal of dust before coating are, however, the principal causes of stripping. Acid igneous rocks generally do not mix well with bitumen as they have a poor ability to absorb it. In contrast, basic igneous rocks such as basalt and dolerite possess a high affinity for bitumen, as does limestone.

Igneous rocks are commonly used for roadstone. Dolerite and basalt have been used extensively. They usually have a high strength and resist abrasion and impact, but their polished stone value generally does not meet motorway specification in Britain, although it is suitable for trunk roads. However, certain dolerites have proved to be susceptible to rapid weathering, for instance, newly exposed fresh dolerite may show extensive signs of disintegration within 18 months (Bell and Jermy, 2000). Swelling due to hydration of secondary montmorillonite plays a part in the rapid breakdown of such dolerite. Rapid breakdown also has occurred in basalts (Haskins and Bell, 1995). These slaking basalts also break down primarily due to the absorption of moisture by secondary smectitic clays within the basalts. Felsite and andesite are much sought after. The coarse-grained igneous rocks such as granite are generally not as suitable as the fine-grained types, as they crush more easily. On the other hand, the very-fine-grained and glassy volcanics are often unsuitable since they produce chips with sharp edges when crushed, and they tend to develop a high polish.

Igneous rocks with a high silica content resist abrasion better than those in which the proportion of ferromagnesian minerals is high, in other words, acid rocks such as rhyolites are harder than basic rocks such as basalts. Some rocks that are the products of thermal metamorphism, such as hornfels and quartzite, because of their high strength and resistance to wear, make good roadstones. In contrast, many rocks of regional metamorphic origin, because of their cleavage and schistosity, are unsuitable. Coarse-grained gneisses offer a similar performance to that of granites. Of the sedimentary rocks, limestone and greywacke frequently are used as roadstone. Greywacke, in particular, has high strength, resists wear and develops a good skid resistance. Some quartz arenites are used, as are gravels. In fact, the use of gravel aggregates is increasing.

Gravels and Sands

Gravel

Gravel deposits usually represent local accumulations, for example, channel fillings. In such instances, they are restricted in width and thickness but may have considerable length. Fan-shaped deposits of gravels or aprons may accumulate at the snouts of ice masses, or blanket deposits may develop on transgressive beaches. The latter type of deposits are usually thin and patchy, whereas the former are frequently wedge shaped.

A gravel deposit consists of a framework of pebbles between which are voids. The voids are rarely empty, being occupied by sand, silt or clay material. River and fluvio-glacial gravels are notably bimodal, the principal mode being in the gravel grade, the secondary in the sand grade. Marine gravels, however, are often unimodal and tend to be more uniformly sorted than fluvial types of similar grade size.

The shape and surface texture of the pebbles in a gravel deposit are influenced by the agent responsible for its transportation and the length of time taken in transport, although shape is also dependent on the initial shape of the fragment, which in turn is controlled by the fracture pattern within the parental rock. Gravel particles can be classified as rounded, irregular, angular, flaky and elongated in shape. Anon (1975a) defines a flakiness index, an elongation index and an angularity number. The flakiness index of an aggregate is the percentage of particles, by weight, whose least dimension (thickness) is less than 0.6 times their mean dimension. The elongation index of an aggregate is the percentage, by weight, of particles whose greatest dimension (length) is greater than 1.8 times their mean dimension. The angularity number is a measure of relative angularity based on the percentage of voids in the aggregate. The least angular aggregates are found to have about 33% voids, and the angularity number is defined as the amount by which the percentage of voids exceeds 33. The angularity number ranges from 0 to about 12. Anon (1975a) also recognized the following types of surface texture, glassy, smooth, granular, rough, crystalline and honeycombed.

The composition of a gravel deposit reflects not only the type of rocks in the source area, but is also influenced by the agents responsible for its formation and the climatic regime in which it was or is being deposited. Furthermore, relief influences the character of a gravel deposit, for example, under low relief, gravel production is small and the pebbles tend to be chemically inert residues such as vein quartz, quartzite, chert and flint. By contrast, high relief and rapid erosion yield coarse, immature gravels. All the same, gravel achieves maturity much more rapidly than does sand under the same conditions. Gravels that consist of only one type of rock fragment are referred to as oligomictic. Such deposits are usually thin and well sorted. Polymictic gravels usually consist of a varied assortment of rock fragments and occur as thick, poorly sorted deposits.

Gravel particles generally possess surface coatings that may be the result of weathering or may represent mineral precipitates derived from circulating groundwater. The latter type of coating may be calcareous, ferruginous, siliceous or, occasionally, gypsiferous. Clay also may form a coating about pebbles. Surface coatings generally reduce the value of gravels for use as concrete aggregate, thick and/or soft and loosely adhering surface coatings are particularly suspect. Clay and gypsum coatings, however, can often be removed by screening and washing. Siliceous coatings tend to react with the alkalies in high-alkali cements and are, therefore, detrimental to concrete.

In a typical gravel pit, the material is dug from the face by a mechanical excavator. This loads the material into trucks or onto a conveyor that transports it to the primary screening and crushing plant. After crushing, the material is further screened and washed. This sorts the gravel into various grades and separates it from the sand fraction. The latter is usually sorted into coarser and finer grades, the coarser is used for concrete and the finer is preferred for mortar. Because gravel deposits are highly permeable, if the water table is high, then the gravel pit will flood. The gravels then have to be worked by dredging. Sea-dredged aggregates are becoming increasingly important.

Sand

The textural maturity of sand varies appreciably. A high degree of sorting, coupled with a high degree of rounding, characterizes mature sand. The shape of sand grains, however, is not greatly influenced by the length of transport. Maturity is reflected in the particle composition of sand, and it has been argued that the ultimate sand is a concentration of pure quartz. This is because the less-stable minerals disappear due to mechanical or chemical breakdown during erosion and transportation or even after sand has been deposited.

Sands are used for building purposes to give bulk to concrete, mortars, plasters and renderings. For example, sand is used in concrete to lessen the void space created by the coarse aggregate. Sand consisting of a range of grade sizes gives a lower proportion of voids than one in which the grains are of uniform size. Indeed, grading is probably the most important property as far as the suitability of sand for concrete is concerned. Anon (1992) recognizes four grades of sand that can produce good-quality concrete. In any concrete mix, consideration should be given to the total specific surface of the coarse and fine aggregates, since this represents the surface that has to be lubricated by the cement paste to produce a workable mix. Poorly graded sands can be improved by adding the missing grade sizes to them, so that high-quality material can be produced with correct blending.

It is generally alleged that sand with rounded particles produces slightly more workable concrete than one consisting of irregularly shaped particles. Sands used for building purposes are usually siliceous in composition and should be as free from impurities as possible. Ideally, they should contain less than 3%, by weight, of silt or clay, since they need a high water content to produce a workable concrete mix. A high water content leads to shrinkage and cracking in concrete on drying. Furthermore, clay and shaley material tend to retard setting and hardening, or they may spoil the appearance of concrete. If sand particles are coated with clay, they form a poor bond with cement and produce a weaker and less durable concrete. The presence of feldspars in sands used in concrete has sometimes given rise to hair cracking, and mica and particles of shale adversely affect the strength of concrete. Organic impurities may affect the setting and hardening properties of cement adversely by retarding hydration and, thereby, reduce its strength and durability. Organic and coaly material also cause popping, pitting and blowing. If iron pyrite occurs in sand, then it gives rise to unsightly rust stains when used in concrete. The salt content of marine sands is unlikely to produce any serious adverse effects in good-quality concrete, although it probably will give rise to efflorescence. Salt can be removed by washing sand.

High-grade quartz sands are used for making silica bricks used for refractory purposes.

Glass sands must have a silica content of over 95% (over 96% for plate glass). The amount of iron oxides present in glass sands must be very low, under 0.05% in the case of clear glass. Uniformity of grain size is another important property, as this means that the individual grains melt in the furnace at approximately the same temperature.

Gravel and Sand Deposits

Scree material or talus accumulates along mountain slopes as a result of freeze–thaw action. Talus is frequently composed of one rock type. The rock debris has a wide range of size distribution, and the particles are angular. Because scree simply represents broken rock material, it is suitable for use as aggregate, if the parent rock is suitable. Such scree deposits, if large enough, only need crushing and screening and therefore are generally more economical than the parent rock.

The composition of a river gravel deposit reflects the rocks of its drainage basin. Sorting takes place with increasing length of river transportation, the coarsest deposits being deposited first, although large fragments can be carried great distances during flood periods. Thus, river deposits possess some degree of uniformity as far as sorting is concerned. Naturally, differences in gradation occur in different deposits within the same river channel but the gradation requirements for aggregate are generally met with or they can be made satisfactory by a small amount of processing. Moreover, as the length of river transportation increases, softer material is progressively eliminated, although in a complicated river system with many tributaries new sediment is being added constantly. Gravel deposits found in river beds are usually characterized by rounded particles. River transportation also roughens the surfaces of pebbles.

River terrace deposits are similar in character to those found in river channels. The pebbles of terrace deposits may possess secondary coatings due to leaching and precipitation. These are frequently of calcium carbonate that does not impair the value of the deposit, but if they are siliceous, then this could react with alkalies in high-alkali cements and therefore could be detrimental to concrete. The longer the period of post-depositional weathering to which a terrace deposit is subjected, the greater is the likelihood of its quality being impaired.

Alluvial cones are found along valleys located at the foot of mountains. They are poorly stratified and contain rock debris with a predominantly angular shape and great variety in size.

Gravels and sands of marine origin are used increasingly as natural aggregates. The winnowing action of the sea leads to marine deposits being relatively clean and uniformly sorted. For the latter reason, these deposits may require some blending. The particles are generally well-rounded, with roughened surfaces. Gravels and sands that occur on beaches normally contain deleterious salts and therefore require vigorous washing. By contrast, much of the salt may have been leached out of the deposits found on raised beaches.

Wind-blown sands are uniformly sorted. They are composed predominantly of well-rounded quartz grains that have frosted surfaces.

Glacial deposits are poorly graded, commonly containing an admixture of boulders and rock flour. What is more, glacial deposits generally contain a wide variety of rock types, and the individual rock fragments are normally subangular. The selective action of physical and chemical breakdown processes is retarded when material is entombed in ice and therefore glacial deposits often contain rock material that is unsuitable for use as aggregate. As a consequence, glacial deposits are usually of limited value as far as aggregate is concerned.

By contrast, fluvio-glacial deposits are frequently worked for aggregate. These deposits were laid down by melt waters that issued from or were associated with bodies of ice. They take the form of eskers, kames and outwash fans (Fig. 3.30). The influence of water on these sediments means that they have undergone a varying degree of sorting. They may be composed of gravels or, more frequently, of sands. The latter are well sorted and may be sharp, thus forming ideal building material.

Lime, Cement and Plaster

Lime is made by heating limestone, including chalk, to a temperature between 1100°C and 1200°C in a current of air, at which point carbon dioxide is driven off to produce quicklime (CaO). Approximately 56 kg of lime can be obtained from 100 kg of pure limestone. Slaking and hydration of quicklime take place when water is added, giving calcium hydroxide. Carbonate rocks vary from place to place both in chemical composition and physical properties so that the lime produced in different districts varies somewhat in its behaviour. Dolostones also produce lime; however, the resultant product slakes more slowly than does that derived from limestones.

Portland cement is manufactured by burning pure limestone or chalk with suitable argillaceous material (clay, mud or shale) in the proportion 3:1. The raw materials are crushed and ground to a powder, and then blended. They are then fed into a rotary kiln and heated to a temperature of over 1800°C. Carbon dioxide and water vapour are driven off, and the lime fuses with the aluminium silicate in the clay to form a clinker. This is ground to a fine powder, and less than 3% gypsum is added to retard setting. Lime is the principal constituent of Portland cement, but too much lime produces a weak cement. Silica constitutes approximately 20% and alumina 5%, both are responsible for the strength of the cement. A high content of the former, however, produces a slow setting, whereas a high content of the latter gives a quick-setting cement. The percentage of iron oxides is low and, in white Portland cement, it is kept to a minimum. The proportion of magnesia (MgO) should not exceed 4%, otherwise the cement is unsound. Similarly, sulphate (SO_4) must not exceed 2.75%. Sulphate-resisting cement is made by the addition of a very small quantity of tricalcium aluminate to normal Portland cement.

When gypsum ($CaSO_4.nH_2O$) is heated to a temperature of 170°C, it loses three quarters of its water of crystallization, becoming calcium sulphate hemi-hydrate, or plaster of Paris. Anhydrous calcium sulphate forms at higher temperatures. These two substances are the chief materials used in plasters. Gypsum plasters have now more or less replaced lime plasters.

Clays and Clay Products

The principal clay minerals belong to the kandite, illite, smectite, vermiculite and palygorskite families. The kandites, of which kaolinite is the chief member, are the most abundant clay minerals. Deposits of kaolin or china clay are associated with granite masses that have undergone kaolinization. The soft china clay is excavated by strong jets of water under high pressure, the material being washed to the base of the pit. This process helps separate the lighter kaolin fraction from the quartz. The lighter material is pumped to the surface of the quarry, where it is fed into a series of settling tanks. These separate mica, which is removed for commercial use, from china clay. Washed china clay has a comparatively coarse size, approximately 20% of the constituent particles being below 0.01 mm in size, accordingly the material is non-plastic. Kaolin is used in the manufacture of white earthenware and stoneware, in white Portland cement and for special refractories.

Ball clays are composed almost entirely of kaolinite and, as between 70 and 90% of the individual particles are below 0.01 mm in size, these clays have a high plasticity. Their plasticity at times is enhanced by the presence of montmorillonite. They contain a low percentage of iron oxide and, consequently, give a light cream colour when burnt. They are used for the manufacture of sanitary ware and refractories.

If a clay or shale can be used to manufacture refractory bricks, then it is termed a fireclay. Such material should not fuse below 1600°C and should be capable of taking a glaze.

Ball clays and china clays are, in fact, fireclays, fusing at 1650 and 1750°C, respectively, however, they are too valuable except for making special refractories. Most fireclays are highly plastic and contain kaolinite as their predominant material. Some of the best fireclays are found beneath coal seams. Indeed, in the United Kingdom, fireclays are restricted almost entirely to the strata of Coal Measures age. The material in a bed of fireclay that lies immediately beneath a coal seam is often of better quality than that found at the base of the bed. Since fireclays represent fossil soils that have undergone severe leaching, they consist chiefly of silica and alumina, and contain only minor amounts of alkalies, lime and iron compounds. This accounts for their refractoriness (alkalies, lime, magnesia and iron oxides in a clay deposit tend to lower its temperature of fusion and act as fluxes). Very occasionally, a deposit contains an excess of alumina and, in such cases, it possesses a very high refractoriness. After the fireclay has been quarried or mined, it is usually left to weather for an appreciable period of time to allow it to breakdown before it is crushed. The crushed fireclay is mixed with water and moulded. Bricks, tiles and sanitary ware are made from fireclay.

Bentonite is formed by the alteration of volcanic ash, the principal clay mineral being either montmorillonite or beidellite. When water is added to bentonite, it swells to many times its original volume to produce a soft gel. Bentonite is markedly thixotropic and this, together with its plastic properties, has given the clay a wide range of uses. For example, it is added to poorly plastic clays to make them more workable and to cement mortars for the same purpose. In the construction industry, it is used as a material for clay grouting, drilling mud, slurry trenches and diaphragm walls.

Evaluation of Mudrocks for Brick Making

The suitability of a raw material for brick making is determined by its physical, chemical and mineralogical character, and the changes that occur when it is fired. The unfired properties, such as plasticity, workability (i.e. the ability of clay to be moulded into shape without fracturing and to maintain its shape when the moulding action ceases), dry strength, dry shrinkage and vitrification range, are dependent on the source material, but the fired properties such as colour, strength, total shrinkage on firing, porosity, water absorption, bulk density and tendency to bloat are controlled by the nature of the firing process. The ideal raw material should possess moderate plasticity, good workability, high dry strength, total shrinkage on firing of less than 10% and a long vitrification range.

The mineralogy of the raw material influences its behaviour during the brick-making process and hence the properties of the finished product (Bell, 1992b). Mudrocks consist of clay minerals and non-clay minerals, mainly quartz. The clay mineralogy varies from one deposit to another. Although bricks can be made from most mudrocks, the varying proportions of different clay minerals have a profound effect on the processing and on the character of the fired brick. Those mudrocks that contain a single predominant clay mineral have a shorter temperature interval between the onset of vitrification and complete fusion than those consisting of a mixture of clay minerals. This is more true of montmorillonitic and illitic mudrocks than those composed chiefly of kaolinite. Also, those clays that consist of a mixture of clay minerals do not shrink as much when fired as those composed predominantly of one type of clay mineral. Mudrocks containing significant amounts of disordered kaolinite tend to have moderate to high plasticity and therefore are easily workable. They produce lean clay materials that undergo little shrinkage during brick manufacture. They also possess a long vitrification range and produce a fairly refractory product. However, mudrocks containing appreciable quantities of well-ordered kaolinite are poorly plastic and less workable. Illitic mudrocks are more plastic and less refractory than those in which disordered kaolinite is dominant, and fire at somewhat lower temperatures. Smectites are the most plastic and least refractory of the clay minerals. They show high shrinkage on drying since they require high proportions of added water to make them workable. As far as the unfired properties of the raw materials are concerned, the non-clay minerals present act mainly as a diluent, but they may be of considerable importance in relation to the fired properties. The non-clay material also may enhance the working properties, for instance, colloidal silica improves workability by increasing the plasticity.

The presence of quartz in significant amounts gives strength and durability to a brick. This is because, during the vitrification period, quartz combines with the basic oxides of the fluxes released from the clay minerals on firing to form glass, which improves the strength. However, as the proportion of quartz increases, the plasticity of the raw material decreases.

The accessory minerals in mudrocks play a significant role in brick making. The presence of carbonates is particularly important and can influence the character of the bricks produced. When heated above 900°C, carbonates break down, yielding carbon dioxide and leaving behind reactive basic oxides, particularly those of calcium and magnesium. The escape of carbon dioxide can cause lime popping or bursting if large pieces of carbonate, for example, shell fragments, are present, thereby pitting the surface of a brick. To avoid lime popping, the material must be ground finely to pass a 20-mesh sieve. The residual lime and magnesia form fluxes that give rise to low-viscosity silicate melts. The reaction lowers the temperature of the brick during firing and hence, unless additional heat is supplied, lowers the firing temperature and shortens the range over which vitrification occurs. The reduction in temperature can result in inadequately fired bricks. If excess oxides remain in a brick, they will hydrate on exposure to moisture, thereby adversely affecting the brick. The expulsion of significant quantities of carbon dioxide can increase the porosity of bricks, reducing their strength. Engineering bricks must be made from a raw material that has low carbonate content.

Sulphate minerals in mudrocks are detrimental to brick making. For instance, calcium sulphate does not decompose within the range of firing temperature of bricks. It is soluble and, if present in trace amounts in the finished brick, causes effluorescence when the brick is exposed to the atmosphere. Soluble sulphates dissolve in the water used to mix the clay. During drying and firing, they often form a white scum on the surface of a brick. Barium carbonate may be added to render such salts insoluble and to prevent scumming.

Iron sulphides, such as pyrite and marcasite, frequently occur in mudrocks. When heated in oxidizing conditions, the sulphides decompose to produce ferric oxide and sulphur dioxide. In the presence of organic matter, oxidation is incomplete, yielding ferrous compounds that combine with silica and basic oxides, if present, to form black glassy spots. This may lead to a black vitreous core being present in some bricks that can significantly reduce strength. If the vitrified material forms an envelope around the ferrous compounds and heating continues until this decomposes, then the gases liberated cannot escape, causing bricks to bloat and distort. Under such circumstances, the rate of firing should be controlled in order to allow gases to be liberated prior to the onset of vitrification. Too high a percentage of pyrite or other iron-bearing minerals gives rise to rapid melting, which can lead to difficulties on firing.

Pyrite, and other iron-bearing minerals such as hematite and limonite, provide the iron that primarily is responsible for the colour of bricks. The presence of other constituents, notably calcium, magnesium or aluminium oxides, tends to reduce the colouring effect of iron oxide, whereas the presence of titanium oxide enhances it. High original carbonate content tends to produce yellow bricks.

Organic matter commonly occurs in mudrock. It may be concentrated in lenses or seams, or be finely disseminated throughout the mudrock. Incomplete oxidation of the carbon upon firing may result in black coring or bloating. Even minute amounts of carbonaceous material can give black coring in dense bricks if it is not burned out. Black coring can be prevented by ensuring that all carbonaceous material is burnt out below the vitrification temperature. This means that if a raw material contains much carbonaceous material, it may be necessary to admit cool air into the firing chamber to prevent the temperature from rising too quickly. On the other hand, the presence of oily material in a clay deposit can be an advantage, for it can reduce the fuel costs involved in brick making. For instance, the Lower Oxford Clay in parts of England contains a significant proportion of oil, so that when heated above approximately 300°C, it becomes almost self-firing.

Mineralogical and chemical information is essential for determining the brick making characteristics of a mudrock. Differential thermal analysis and thermogravimetric analysis can identify clay minerals in mudrocks, but provide only very general data on relative abundance. X-ray diffraction methods are used to determine the relative proportions of clay and other minerals present (Fig. 6.7a). The composition of the clay minerals present can also be determined by plotting ignition loss against moisture absorption (Fig. 6.7b). The moisture absorption characterizes the type of clay mineral present, whereas ignition loss provides some indication of the quantity present.

Sufficient quantities of suitable raw material must be available at a site before a brick pit can be developed. The volume of suitable mudrock must be determined as well as the amount of waste, that is, the overburden and unsuitable material within the sequence that

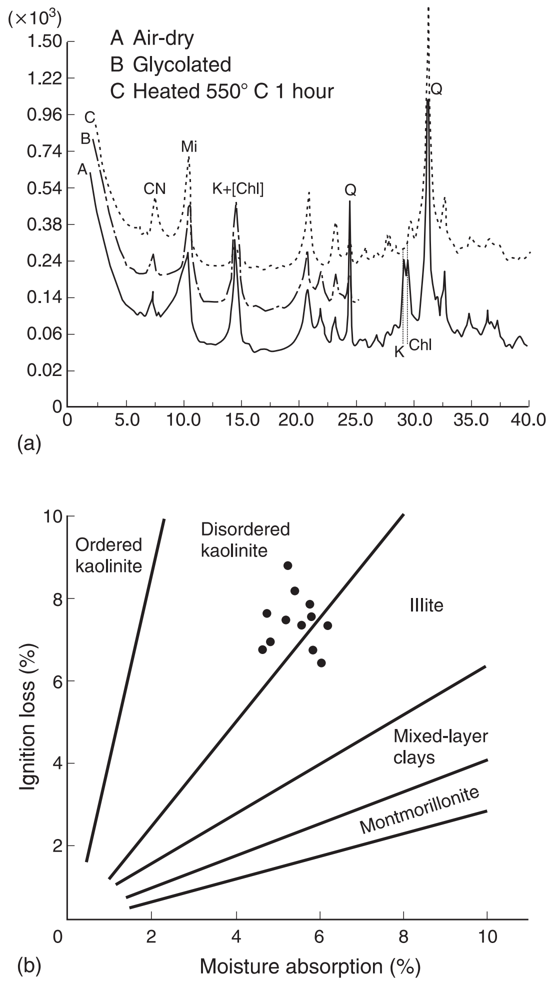


Figure 6.7

(a) X-ray diffraction traces of a sample of Coal Measures shale used for brick making. Chl = chlorite; Mi = mica; K = kaolinite; Q = quartz. (b) Clay mineral determination using Keeling's method.

is to be extracted (Bell, 1992b). The first stages of the investigation are topographical and geological surveys, followed by a drillhole programme. This leads to a lithostratigraphic and structural evaluation of the site. It also should provide data on the position of the water table and the stability of the slopes that will be produced during excavation of the brick pit.

Bricks

Mudrocks are dug from a brick pit by a variety of mechanical excavators, such as face shovels, draglines and continuous strippers (Fig. 6.8). The material is then stockpiled to allow weathering to aid its breakdown. It is then crushed and sieved before being moulded, dried and fired. There are four main methods of brick production in the United Kingdom, namely, the wire-cut process, the semi-dry pressed method, the stiff plastic method and moulding by hand or machine. One of the distinguishing factors between these methods is the moisture content of the raw material when the brick is fashioned. This varies from as little as 10% in the case of semi-dry pressed bricks to 25% or more in hand-moulded bricks. Hence, the natural moisture content of a mudrock can have a bearing on the type of brick-making operation.

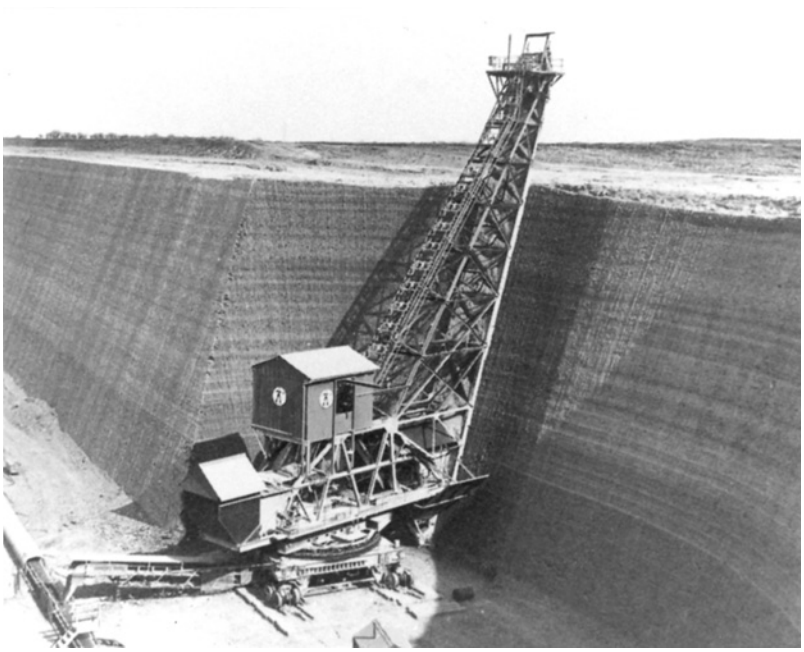


Figure 6.8

A continuous stripper working the Lower Oxford Clay at Whittlesey, near Peterborough, England.

For example, many mudrocks have a natural moisture content in excess of 15% and therefore are unsuitable for the semi-dry pressed or even the stiff plastic methods of production unless they are dried before moulding.

Three stages can be recognized in brick burning. During the water-smoking stage, which takes place up to approximately 600°C, water is given off. Pore water and the water with which the clay was mixed are driven off at about 110°C, whereas water of hydration disappears between 400°C and 600°C. The next stage is that of oxidation, during which the combustion of carbonaceous matter and the decomposition of iron pyrite takes place, and carbon dioxide, sulphur dioxide and water vapour are given off. The final stage is that of vitrification. Above 800°C, the centre of the brick gradually develops into a highly viscous mass, the fluidity of which increases as the temperature rises. The components are now bonded together by the formation of glass. Bricks are fired at temperatures around 1000°C to 1100°C for about 3 days. The degree of firing depends on the fluxing oxides, principally H_2O, Na_2O, CaO and Fe_2O_3. Mica is one of the chief sources of alkalies in clay deposits. Because illites are more intimately associated with micas than kaolinites, illitic clays usually contain a higher proportion of fluxes and are therefore less refractory than kaolinitic clays.

The strength of the brick depends largely on the degree of vitrification. Theoretically, the strength of bricks made from mudrocks containing fine-grained clay minerals such as illite should be higher than those containing the coarser grained kaolinite. Illitic clays, however, vitrify more easily, and there is a tendency to underfire, particularly if they contain fine-grained calcite or dolomite. Kaolinitic clays are much more refractory and can stand harder firing, greater vitrification is therefore achieved.

Permeability also depends on the degree of vitrification. Mudrocks containing a high proportion of clay minerals produce less permeable products than clays with a high proportion of quartz, but the former types of clays may have a high drying shrinkage.

The colour of a mudrock prior to burning gives no indication of the colour it will have after leaving the kiln. The iron content of the raw material, however, is important in this respect. For instance, as there is less scope for iron substitution in kaolinite than in illite, this often means that kaolinitic clays give a whitish or pale yellow colour on firing, whereas illitic clays generally produce red or brown bricks. More particularly, clay possessing about 1% of iron oxides when burnt may produce a cream or light yellow colour, 2 to 3% gives buff, and 4 to 5% red. Other factors, however, must be taken into account. For instance, a clay containing 4% Fe_2O_3 under oxidizing conditions burns pink below 800°C, turns brick red at about 1000°C and, at 1150°C, as vitrification approaches completion, it adopts a deep red colour. By contrast, under reducing conditions, ferrous silicate develops, and the clay product has a blackish colour. Reducing conditions are produced if carbonaceous material is present in the clay, or they may be brought about by the addition of coal or sawdust to the clay before it is burnt. Blue bricks are also produced under reducing conditions. The clay should contain about 5% iron, together with lime and alkalies. An appreciable amount of lime in clay tends to lighten the colour of the burnt product, for example, 10% of lime does not affect the colour at 800°C, but at higher temperatures, with the formation of calcium ferrites, a cream colour is developed. This occurs in clays with 4% of Fe_2O_3 or less. The presence of manganese in clay may impart a purplish shade to the burned product.

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Site Investigation

The general objective of a site investigation is to assess the suitability of a site for the proposed purpose. As such, it involves exploring the ground conditions at and below the surface (Anon, 1999). It is a prerequisite for the successful and economic design of engineering structures and earthworks. Accordingly, a site investigation also should attempt to foresee and provide against difficulties that may arise during construction because of ground and/or other local conditions. Indeed, investigation should not cease once construction begins. It is essential that the prediction of ground conditions that constitute the basic design assumption be checked as construction proceeds and designs modified accordingly if conditions are revealed to be different from those predicted. The investigation of a site for an important structure requires exploration and sampling of strata likely to be significantly affected by the structural load. Data appertaining to groundwater conditions, extent of weathering, and discontinuity pattern in rock masses are also important. In some areas there are special problems that need investigating, for example, potential subsidence in areas of shallow abandoned mine workings and contaminated ground. What is more, as Culshaw (2005) pointed out, the rapid development of information technology and the digitization of increasing amounts of geological data now means that it often is possible to produce three-dimensional (3D) special models of the shallow subsurface.

The complexity of a site investigation depends upon the nature of the ground conditions and the type of engineering structure. More complicated ground conditions and sensitive large engineering structures require more rigorous investigation of the ground conditions. Although a site investigation usually consists of three stages, namely a desk study, a preliminary reconnaissance and a site exploration, there must be a degree of flexibility in the procedure since no two sites are the same.

Desk Study and Preliminary Reconnaissance

A desk study is undertaken as the first stage of a site investigation in order to make an initial assessment of the ground conditions and to identify, if possible, any potential geotechnical problems (Herbert et al., 1987). In other words, the objective of a desk study is to examine available archival records, literature, maps, imagery and photographs relevant to the area or site concerned to ascertain a general picture of the existing geological conditions prior to a field investigation, that is, to begin the process of constructing what Fookes (1997) referred to as the geological model. The effort expended in any desk study depends on the complexity and size of the proposed project and on the nature of the ground conditions. Detailed searches for information, however, can be extremely time consuming and may not be justified for small schemes at sites where the ground conditions are relatively simple. In such cases, a study of the relevant topographical and geological maps and memoirs, and possibly aerial photographs, may suffice. On large projects, literature and map surveys may save time and thereby reduce the cost of the site investigation programme. The data obtained during such searches should help the planning of the subsequent site exploration and prevent duplication of effort. In some parts of the world, however, little or no literature or maps are available.

Topographical, geological and soil maps can provide valuable information that can be used during the planning stage of a construction operation. The former are particularly valuable when planning routeways. Geological maps afford a generalized picture of the geology of an area. Generally, the stratum boundaries and positions of the structural features, especially faults, are interpolated. As a consequence, their accuracy cannot always be trusted. Map memoirs may accompany maps, and these provide a detailed survey of the geology of the area in question.

From the engineer's point of view, one of the shortcomings of conventional geological maps is that the boundaries are stratigraphical and more than one type of rock may be included in a single mappable unit. What is more, the geological map is lacking in quantitative information that the engineer requires, concerning such facts as the physical properties of the rocks and soils, the amount and degree of weathering, the hydrogeological conditions, etc. However, information such as that relating to the distribution of superficial deposits, landslipped areas and potential sources of construction materials frequently can be obtained from geological maps. A geological map also can be used to indicate those rocks or soils that could be potential sources of groundwater. Now many geological surveys are producing hazard maps, environmental geology maps and engineering geology maps that provide data more relevant to engineers and planners. Such maps represent an attempt to make geological information more understandable by the non-geologist. Frequently, because it is impossible to represent all environmental or engineering data on one map, a series of thematic maps, each of a different topic, is incorporated into a report on a given area. Smith and Ellison (1999) have provided a review of applied geological maps for planning and development. Their review describes various ways by which geological data can be represented on maps.

A desk study for the planning stage of a project can encompass a range of appraisals from the preliminary rapid response to the comprehensive. Nonetheless, there are a number of common factors throughout this spectrum that need to be taken into account. These are summarised in Table 7.1, from which it can be concluded that an appraisal report typically includes a factual and interpretative description of the surface and geological conditions, information on previous site usage, a preliminary assessment of the suitability of the site for the planned development, an identification of potential constraints and provisional recommendations with regard to ground engineering aspects. A desk study also can reduce the risk of encountering unexpected ground conditions that could adversely affect the financial viability of a project. However, a desk study should not be regarded as an alternative to a ground exploration for a construction project.

The preliminary reconnaissance involves a walk over the site. The factors that should be taken into account depend on the nature of the site and the project but, where possible, note can be taken of the distribution of the soil and rock types present, the relief of the ground, the surface drainage and associated features, any actual or likely landslip areas, ground cover and obstructions, earlier uses of the site such as tipping or evidence of underground workings etc. The inspection should not be restricted to the site but should examine adjacent areas to see how they affect or will be affected by construction on the site in question.

The importance of the preliminary investigation is that it should assess the suitability of the site for the proposed works, and if it is suitable, it will form the basis upon which the site exploration is planned. The preliminary reconnaissance also allows a check to be made on any conclusions reached in the desk study.

Remote Sensing

Remote imagery and aerial photographs prove to be invaluable during the planning and reconnaissance stages of certain projects. The information they provide can be transposed to a base map, which is checked during fieldwork. The data also can be used in geographical information systems.

Remote sensing involves the identification and analysis of phenomena on the Earth's surface by using devices borne by aircraft or spacecraft. Most techniques used in remote sensing depend on recording energy from part of the electromagnetic spectrum, ranging from gamma rays through the visible spectrum to radio waves. The scanning equipment used measures both emitted and reflected radiation, and the employment of suitable detectors and filters permits the measurement of certain spectral bands. Two of the main systems of remote sensing are infrared linescan (IRLS) and side-looking airborne radar (SLAR). Signals from several bands of the spectrum can be recorded simultaneously by multi-spectral scanners. Lasers are used in remote sensing.

Table 7.1. Summary contents of engineering geological desk study appraisals (after Herbert et al., 1987). With kind permission of the Geological Society

Item	Content and main points of relevance
Introduction	Statement of terms of reference and objectives, with indication of any limitations. Brief description of nature of project and specific ground-orientated proposals. Statement of sources of information on which appraisal is based
Ground conditions	Description of relevant factual information. Identification of any major features that might influence scheme layout, planning or feasibility
Site description and topography	Descriptions of existing surface conditions from study of topographic maps and actual photographs, and also from site walkover inspection (if possible)
Engineering history	Review of information on previous surface conditions and usage (if different from present) based on study of old maps, photographs, archival records and related to any present features observed during site walkover. Identification of features such as landfill zones, mine workings, pits and quarries, sources of contamination, old water courses, etc.
Engineering geology	Description of subsurface conditions, including any information on groundwater, from study of geological maps and memoirs, previous site investigation reports and any features or outcrops observed during site walkover. Identification of possible geological hazards, e.g. buried channels in alluvium, solution holes in chalk and limestone, swelling/shrinkable clays
Provisional assessment of site suitability	Summary of main engineering elements of proposed scheme, as understood. Comments on suitability of site for proposed development, based on existing knowledge
Provisional land classification	Where there is significant variation in ground conditions or assessed level of risk, subdivision of the site into zones of high and low risk, and any intermediate zones. Comparison of various risk zones with regard to the likely order of cost and scope of subsequent site investigation requirements, engineering implications, etc.
Provisional engineering comments	Statement of provisional engineering comments on such aspects as foundation conditions and which methods appears most appropriate for structural foundations and ground slabs, road pavement subgrade conditions, drainage, excavatability of soils and rocks, suitability of local borrow materials for use in construction, slope stability considerations, nature and extent of any remedial works, temporary problems during construction
Recommendations for further work	Proposals for phased ground investigation, with objectives, requirements and estimated budget costs

Infrared linescanning is dependent upon the fact that all objects emit electromagnetic radiation generated by the thermal activity of their component atoms. Identification of grey tones is the most important aspect as far as the interpretation of thermal imagery is concerned, since these provide an indication of the radiant temperatures of a surface. Warm areas give rise to light, and cool areas to dark tones. The data can be processed in colour as well as black and white, colours substituting for grey tones. Relatively cool areas are depicted as purple and relatively warm areas as red on a positive print. Thermal inertia is important in this respect since rocks with high thermal inertia, such as dolostone or quartzite, are relatively cool during the day and warm at night. Rocks and soils with low thermal inertia, for example, shale, gravel or sand, are warm during the day and cool at night. The variation in temperature of materials with high thermal inertia during the daily cycle is much less than those with low thermal inertia. Because clay soils possess relatively high thermal inertia, they appear warm in pre-dawn imagery, whereas sandy soils, because of their relatively low thermal inertia, appear cool. The moisture content of a soil influences the image produced, that is, soils that possess high moisture content appear cool irrespective of their type. Consequently, high moisture content may mask differences in soil types. Fault zones often are picked out because of their higher moisture content. Similarly, the presence of old landslides frequently can be discerned because their moisture content differs from that of their surroundings.

Texture also can help interpretation. For instance, outcrops of rock may have a rough texture due to the presence of bedding or jointing, whereas soils usually give rise to a relatively smooth texture. However, where soil cover is less than 0.5 m, the rock structure usually is observable on the imagery since deeper, more moist soil occupying discontinuities gives a darker signature. Free-standing bodies of water usually are readily visible on thermal imagery, however, the thermal inertia of highly saturated organic deposits may approach that of water masses, the two therefore may prove difficult to distinguish at times.

In side-looking airborne radar, short pulses of energy, in a selected part of the radar waveband are transmitted sideways to the ground from antennae on both sides of an aircraft. The pulses strike the ground and are reflected back to the aircraft. The reflected pulses are transformed into black-and-white photographs. Mosaics of photographs are suitable for the identification of regional geological features and for preliminary identification of terrain units. Lateral overlap of radar cover can give a stereoscopic image, which offers a more reliable assessment of the terrain and can provide appreciable detail of landforms. The wavelengths used in SLAR are not affected by cloud cover. This is particularly important in equatorial regions, which are rarely free of cloud.

The large areas of the ground surface that satellite images can cover give a regional physiographic setting and permit the distinction of various landforms. Accordingly, such imagery can provide a geomorphological framework from which a study of the component landforms is possible. The character of the landforms may afford some indication of the type of material of which they are composed, and geomorphological data aid the selection of favourable sites for investigation on larger-scale aerial surveys.

The value of space imagery is important where existing map coverage is inadequate. For example, it can be of use for the preparation of maps of terrain classification, interpretation of geological structure, geomorphological studies, regional engineering soil maps, maps used for route selection, regional inventories of construction materials, groundwater studies, and inventories of drainage networks and catchment areas (Sabins, 1996). A major construction project is governed by the terrain. In order to assess the ground conditions, it is necessary to make a detailed study of all the photo-pattern elements that comprise the landforms on the satellite imagery. Important evidence relating to soil types, or surface or subsurface conditions may be provided by erosion patterns, drainage characteristics or vegetative cover. Engineering soil maps frequently are prepared on a regional basis for both planning and location purposes in order to minimize construction costs, the soils being delineated for the landforms within the regional physiographic setting.

Satellite imagery has improved in its resolution over time so that its use has extended from regional geological mapping and mineral exploration to larger-scale geomorphological mapping and geohazard identification. High-resolution airborne geophysical surveys involving magnetic, gamma spectrometry and very-low-frequency electromagnetic sensors are improving the ability to locate, for example, landfills with high ferrous contents, contaminated sites and abandoned mine sites. Such surveys provide rapid comprehensive data coverage, which permits focused confirmatory ground coverage. This is advantageous when investigating hazardous sites.

Later-generation LANDSAT satellites carry an improved imaging system called thematic mapper (TM) as well as a multi-spectral scanner (MSS). Thermal mapper images have a spatial resolution of 30 m and excellent spectral resolution. Generally, TM bands are processed as normal and infrared colour images. Data gathered by Landsat TM are available as CD-ROMS, which can be read and processed by computers. The weakest point in the system is the lack of adequate stereovision capability, however, a stereomate of a TM image can be produced with the help of a good digital elevation model.

Radar and laser sensors on airborne platforms are being used to produce high-resolution (centimetre to metre) digital terrain models. The light detecting and ranging (LIDAR) system sends a laser pulse from an airborne platform to the ground and measures the speed and intensity of the returning signal. From this, changes in ground elevation can be mapped. Radar systems use radars rather than lasers to achieve the same end. The satellite technique known as permanent scatterer interferometry (PSInSAR) uses radar data collected by satellites 800 km out in space. The PSInSAR method exploits a dense network of “natural” reflectors that can be any hard surface such as a rock outcrop, a building wall or roof or a road kerb. These reflectors are visible to the radar sensor over many years. Permanent scatterer interferometry produces maps showing rates of displacement, accurate to a few millimetres per year, over time periods, currently up to a decade long. The process provides millimetric displacement histories for each reflector point across the entire time period analysed, as calculated at every individual radar scene acquisition. Hence, small incremental ground movements can be detected.

Aerial Photographs

The amount of useful information that can be obtained from aerial photographs varies with the nature of the terrain and the type and quality of the photographs. A study of aerial photographs allows the area concerned to be divided into topographical and geological units, and enables the engineering geologist to plan fieldwork and to select locations for sampling. This should result in a shorter, more profitable period in the field.

Aerial photographs are being digitized and distributed on CD-ROMS that are compatible with desktop computers and image processing software. Orthophotographs are aerial photographs that have been scanned into digital format and computer processed so that radial distortion is removed. These photographs have a consistent scale and therefore may be used in the same ways as maps.

Examination of consecutive pairs of aerial photographs with a stereoscope allows observation of a 3D image of the ground surface. The 3D image means that heights can be determined and contours can be drawn, thereby producing a topographic map. However, the relief presented in this image is exaggerated, and therefore slopes appear steeper than they actually are. Nonetheless, this helps the detection of minor changes in slope and elevation. Unfortunately, exaggeration proves a definite disadvantage in mountainous areas, as it becomes difficult to distinguish between steep and very steep slopes. A camera with a longer focal lens reduces the amount of exaggeration, and therefore its use may prove preferable in such areas. Digital photogrammetric methods use digital images and a computer instead of a photogrammetric plotter to derive digital elevation models (DEMs) with the advantage that various aspects of the measurement process can be automated (Chandler, 2001).

There are four main types of film used in normal aerial photography, namely black and white, infrared monochrome, true colour and false colour. Black-and-white film is used for topographic survey work and for normal interpretation purposes. The other types of film are used for special purposes. For example, infrared monochrome film makes use of the fact that near-infrared radiation is strongly absorbed by water. Accordingly, it is of particular value when mapping shorelines, the depth of shallow underwater features and the presence of water on land, as for instance, in channels, at shallow depths underground or beneath vegetation. Furthermore, it is more able to penetrate haze than conventional photography. True colour photography generally offers much more refined imagery. As a consequence, colour photographs have an advantage over black and white ones as far as photogeological interpretation is concerned. False colour is the term frequently used for infrared colour photography. False colour provides a more sensitive means of identifying exposures of bare grey rocks than any other type of film. Lineaments, variations in water content in soils and rocks and changes in vegetation that may not be readily apparent on black-and-white photographs often are clearly depicted by false colour. A summary of the types of geological information that can be obtained from aerial photographs is given in Table 7.2.

Site Exploration – Direct Methods

The aim of a site exploration is to try to determine and thereby understand the nature of the ground conditions on site and those of its surroundings (Clayton et al., 1996). The extent to which this stage of a site investigation is carried depends, to some extent, upon the size and importance of the construction operation. The site exploration must be concluded by a report embodying the findings, which can be used for design purposes. This should contain geological plans of the site with accompanying sections, thereby conveying a 3D picture of the subsurface strata.

The scale of the mapping will depend on the engineering requirement, the complexity of the geology, and the staff and time available. Rock and soil types should be mapped according to their lithology and, if possible, presumed physical behaviour, that is, in terms of their engineering classification, rather than age. Geomorphological conditions, hydrogeological conditions, landslips, subsidences, borehole and field-test information all can be recorded on geotechnical maps. Particular attention should be given to the nature of the superficial deposits and, where present, made-over ground.

There are no given rules regarding the location of boreholes or drillholes, or the depth to which they should be sunk. This depends upon the geological conditions and the type of project concerned. The information provided by the desk study, the preliminary reconnaissance and from any trial trenches should provide a basis for the initial planning and layout of the borehole or drillhole programme. Holes should be located so as to detect the geological sequence and structure. Obviously, the more complex this is, the greater the number of holes needed. In some instances, it may be as well to start with a widely spaced network of holes. As information is obtained, further holes can be put down if and where necessary.

Table 7.2. Types of photogeological investigation

Structural geology	Mapping and analysis of folding. Mapping of regional fault systems and recording any evidence of recent fault movements. Determination of the number and geometry of joint systems
Rock types	Recognition of the main lithological types (crystalline and sedimentary rocks, unconsolidated deposits)
Soil surveys	Determining main soil type boundaries, relative permeabilities and periglacial studies
Topography	Determination of relief and landforms. Assessment of stability of slopes, detection of old landslides
Stability	Slope instability (especially useful in detecting old failures that are difficult to appreciate on the ground) and rock fall areas, quick clays, loess, peat, mobile sand, soft ground and features associated with old mine workings
Drainage	Outlining of catchment areas, steam divides, surface run-off characteristics, areas of subsurface drainage such as karstic areas, especially of cavernous limestone as illustrated by surface solution features; areas liable to flooding. Tracing swampy ground, perennial or intermittent streams, and dry valleys. Levees and meander migration. Flood control studies. Forecasting effect of proposed obstructions. Run-off characteristics. Shoals, shallow water, stream gradients and widths
Erosion	Areas of wind, sheet and gully erosion, excessive deforestation, stripping for opencast work, coastal erosion
Groundwater	Outcrops and structure of aquifers. Water bearing sands and gravels. Seepages and springs, possible productive fracture zones. Sources of pollution. Possible recharge sites
Reservoirs and dam sites	Geology of reservoir site, including surface permeability classification. Likely seepage problems. Limit of flooding and rough relative values of land to be submerged. Bedrock gullies, faults and local fracture pattern. Abutment characteristics. Possible diversion routes. Ground needing clearing. Suitable areas for irrigation
Materials	Location of sand and gravel, clay, rip-rap, borrow and quarry sites with access routes
Routes	Avoidance of major obstacles and expensive land. Best graded alternatives and ground conditions. Sites for bridges. Pipe and power line reconnaissance. Best routes through urban areas
Old mine workings	Detection of shafts and shallow abandoned workings, subsidence features

Exploration should be carried out to a depth that includes all strata likely to be significantly affected by structural loading. As far as soils are concerned, experience has shown that damaging settlement usually does not take place when the added stress in the soil due to the weight of a structure is less is less than 10% of the effective overburden stress. It therefore would seem logical to sink boreholes on compact sites to depths where the additional stress does not exceed 10% of the stress due to the weight of the overlying strata. It must be borne in mind that if a number of loaded areas are in close proximity the effect of each is additive. Under certain special conditions, holes may have to be sunk more deeply as, for example, when voids due to abandoned mining operations are suspected or when it is suspected that there are highly compressible layers, such as interbedded peats, at depth. If possible, boreholes should be taken through superficial deposits to rockhead. In such instances, adequate penetration of the rock should be specified to ensure that isolated boulders are not mistaken for the solid formation.

The results from a borehole or drillhole should be documented on a log (Fig. 7.1). Apart from the basic information such as number, location, elevation, date, client, contractor and engineer responsible, the fundamental requirement of a borehole log is to show how the sequence of strata changes with depth. Individual soil or rock types are presented in symbolic form on a log. The material recovered must be described adequately, and in the case of rocks frequently include an assessment of the degree of weathering, fracture index and relative strength. The type of boring or drilling equipment should be recorded, the rate of progress made being a significant factor. The water level in the hole and any water loss, when it is used as a flush during rotary drilling, should be noted, as these reflect the mass permeability of the ground. If any in situ testing is done during boring or drilling operations, then the type(s) of test and the depth at which it/they were carried out must be recorded. The depths from which samples are taken must be noted. A detailed account of the logging of cores for engineering purposes is provided by Anon (1970). Description and classification of soils, and of rocks and rock masses, can be found in Anon (1999), while a description and classification of weathered rocks is given in Anon (1995) and of discontinuities in Barton (1978).

Direct observation of strata down a hole, of discontinuities and cavities can be undertaken by cameras or closed-circuit television equipment, and drillholes can be viewed either radially or axially. The camera can be used in boreholes or drillholes down to a minimum diameter of 100 mm. Remote focusing for all heads and rotation of the radial head through 360° are controlled from the surface. The television heads have their own light source. Focussing, light intensity, rotation and digital depth control on the image are made by means of a surface control unit and the image is recorded on standard VHS format video tape. Colour changes in rocks can be detected as a result of the varying amount of light reflected from the drillhole walls. Discontinuities appear as dark areas because of the non-reflection of light. However, if the drillhole is deflected from the vertical, variations in the distribution of light may result in some lack of picture definition.

Subsurface Exploration in Soils

The simplest method whereby data relating to subsurface conditions in soils can be obtained is by hand augering. Soil samples that are obtained by augering are badly disturbed

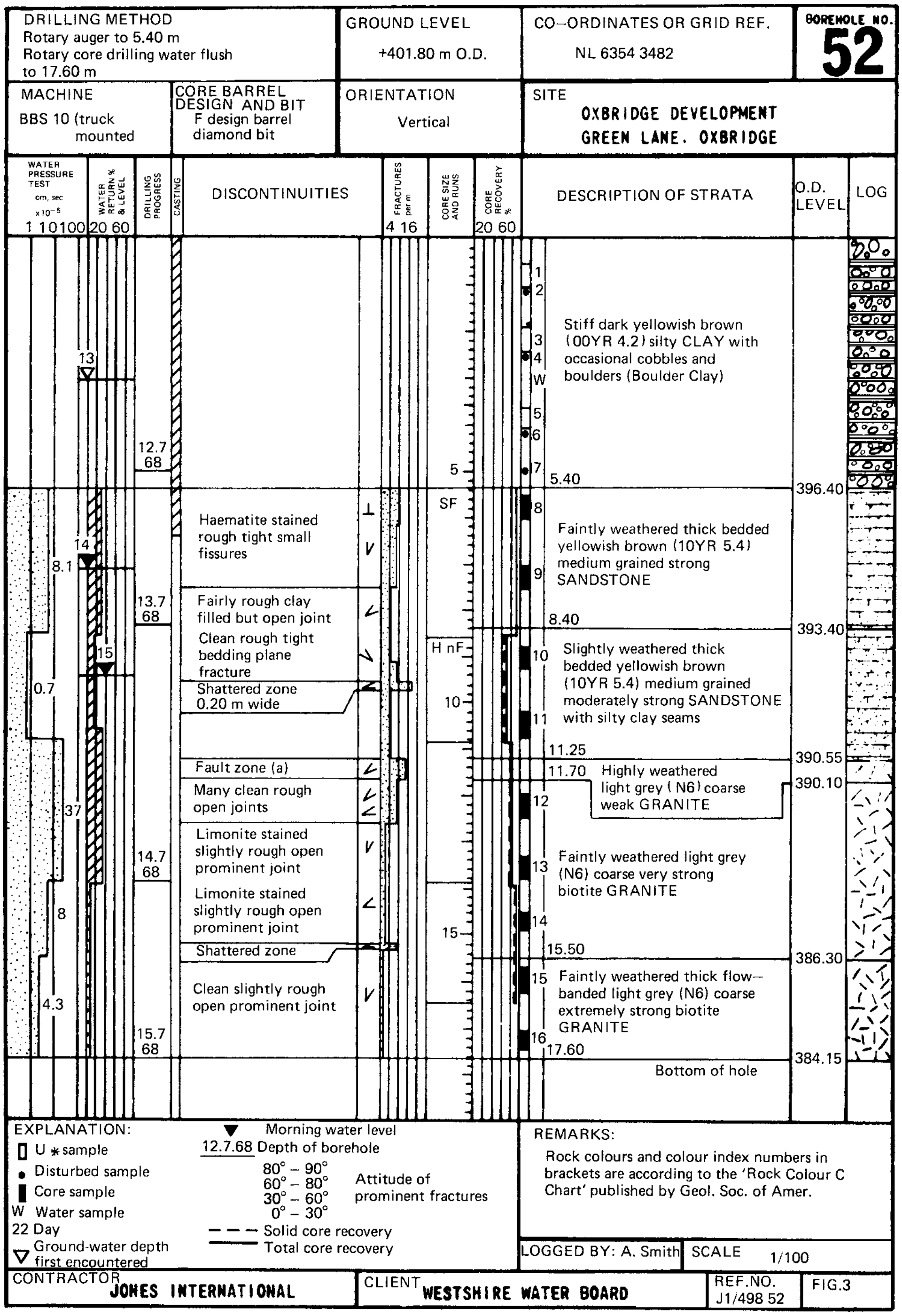


Figure 7.1

Drillhole log. With kind permission of the Geological Society.

and invariably some amount of mixing of soil types occurs. Critical changes in ground conditions therefore are unlikely to be located accurately. Even in very soft soils it may be very difficult to penetrate more than 7 m with hand augers.

Power augers are available as solid stem or hollow stem both having an external continuous helical flight. The later are used in those soils in which the borehole does not remain open. The hollow stem can be sealed at the lower end with a combined plug and cutting bit that is removed when a sample is required. Hollow-stem augers are useful for investigations where the requirement is to locate bedrock beneath overburden. Solid-stem augers are used in stiff clays that do not need casing, however, if an undisturbed sample is required, then they have to be removed. Disturbed samples taken from auger holes often are unreliable. In favourable ground conditions, such as firm and stiff homogeneous clays, auger rigs are capable of high output rates. The development of large earth augers and patent piling systems have made it is possible to sink 1 m diameter boreholes in soils more economically than previously. The ground conditions can be inspected directly from such holes. Depending on the ground conditions, the boreholes may be unlined, lined with steel mesh or cased with steel pipe. In the latter case, windows are provided at certain levels for inspection and sampling.

Pits and trenches allow the ground conditions in soils and highly weathered rocks to be examined directly, although they are limited as far as their depth is concerned. Trenches can provide a flexible, rapid and economic method of obtaining information. Groundwater conditions and stability of the sides obviously influence whether or not they can be excavated, and safety must at all times be observed, necessitating shoring the sides. Pits are expensive and should be considered only if the initial subsurface survey has revealed any areas of special difficulty. The soil conditions in pits and trenches can be mapped or photographed throughout. Undisturbed, as well as disturbed, samples can be collected where necessary.

The light cable and tool boring rig is used for investigating soils (Fig. 7.2). The hole is sunk by repeatedly dropping one of the tools into the ground. A power winch is used to lift the tool, suspended on a cable, and by releasing the clutch of the winch the tool drops and cuts into the soil. Once a hole is established, it is lined with casing, the drop tool operating within the casing. This type of rig usually is capable of penetrating about 60 m of soil, and in doing so the size of the casing in the lower end of the borehole is reduced. The basic tools are the shell and the clay cutter, which are essentially open-ended steel tubes to which cutting shoes are attached. The shell, which is used in granular soils, carries a flap valve at its lower end, which prevents the material from falling out on withdrawal from the borehole. The material is retained in the cutter by the adhesion of the clay.

For boring in stiff clays the weight of the clay cutter may be increased by adding a sinker bar. In very stiff clays, a little water often is added to assist boring progress. This must be done

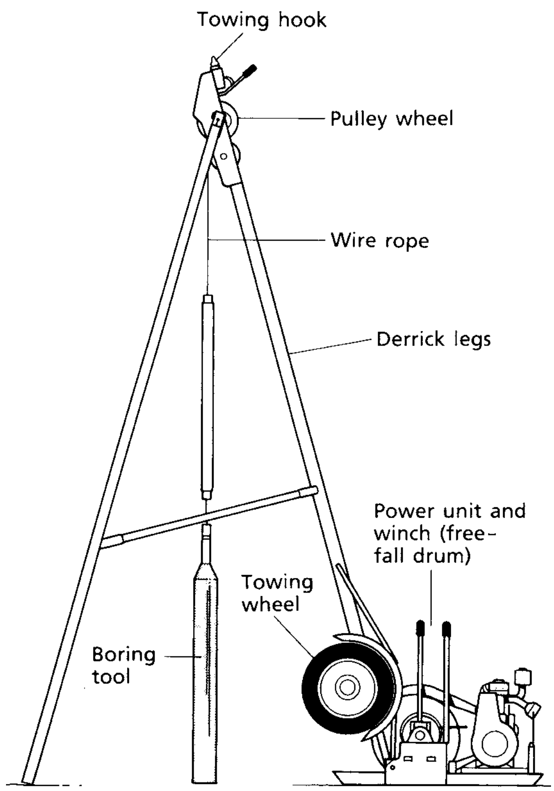


Figure 7.2

Light cable and tool percussion boring rig.

with caution so as to avoid possible changes in the properties of any soil about to be sampled. Furthermore, in such clays, the borehole often can be advanced without lining, except for a short length at the top to keep the hole stable. If cobbles or small boulders are encountered in clays, particularly tills, then these can be broken by using heavy chisels.

When boring in soft clays, although the hole may not collapse, it tends to squeeze inwards and to prevent the cutter operating; the hole therefore must be lined. The casing is driven in and winched out, however, in difficult conditions it may have to be jacked out. Casing tubes have internal diameters of 150, 200, 250 and 300 mm, the most commonly used sizes being 150 and 200 mm (the large sizes are used in coarse gravels).

The usual practice is to bore ahead of the casing for about 1.5 m (the standard length of a casing section) before adding a new section of casing and surging it down. The reason for surging the casing is to keep it “free” in the borehole so that it can be extracted more easily on completion. When the casing can no longer be advanced by surging, smaller diameter casing is introduced. If the hole is near its allotted depth the casing may be driven into the ground for quickness. Where clay occurs below coarse deposits, the casing used as a support in the coarse soils is driven a short distance into the clay to create a seal and the shell is used to remove any water that might enter the borehole.

Boreholes in sands or gravels almost invariably require lining. The casing should be advanced with the hole or overshelling is likely to occur, that is, the sides collapse and prevent further progress. Because of the mode of operation of the shell, the borehole should be kept full of water so that the shell may operate efficiently. Where coarse soils are water-bearing, all that is necessary is for the water in the borehole to be kept topped up. If flow of water occurs, then it should be from the borehole to the surrounding soil. However, if water is allowed to flow into the borehole, piping probably will occur. Piping usually can be avoided by keeping the head of water in the borehole above the natural head. To overcome artesian conditions, the casing should be extended above ground and kept filled with water. The shell generally cannot be used in highly permeable coarse gravels since it usually is impossible to maintain a head of water in the borehole. Fortunately, these conditions often occur at or near ground level, and the problem can sometimes be overcome by using an excavator to open a pit either to the water table or to a depth of 3 to 4 m. Casing can then be put in place, the pit backfilled and boring then can proceed. Another method of penetrating gravels and cobbles above the water table is to employ a special grab with a heavy tripod and winch and casing of 400 mm diameter or greater.

Rotary attachments are available that can be used with light cable and tool rigs. However, they are much less powerful than normal rotary rigs and tend to be used only for short runs as, for example, to prove rockhead at the base of a borehole.

In the wash boring method, the hole is advanced by a combination of chopping and jetting the soil or weak or weathered rock, the cuttings thereby produced being washed from the hole by the water used for jetting (Fig. 7.3). The method cannot be used for sampling, and therefore its primary purpose is to sink the hole between sampling positions. When a sample is required, the bit is replaced by a sampler. Nevertheless, some indication of the type of ground penetrated may be obtained from the cuttings carried to the surface by the wash water, from the rate of progress made by the bit or from the colour of the wash water. Several types of chopping bits are used. Straight and chisel bits are used in sands, silts, clays and very soft rocks, while cross bits are used in gravels and soft rocks. Bits are available with either the jetting points facing upwards or downwards. The former types are better at cleaning the base of the hole than are the latter. The wash boring method may be used in both cased and uncased holes. Casing obviously has to be used in coarse soils to avoid the sides of the hole from collapsing. Although this method of boring commonly is used in the

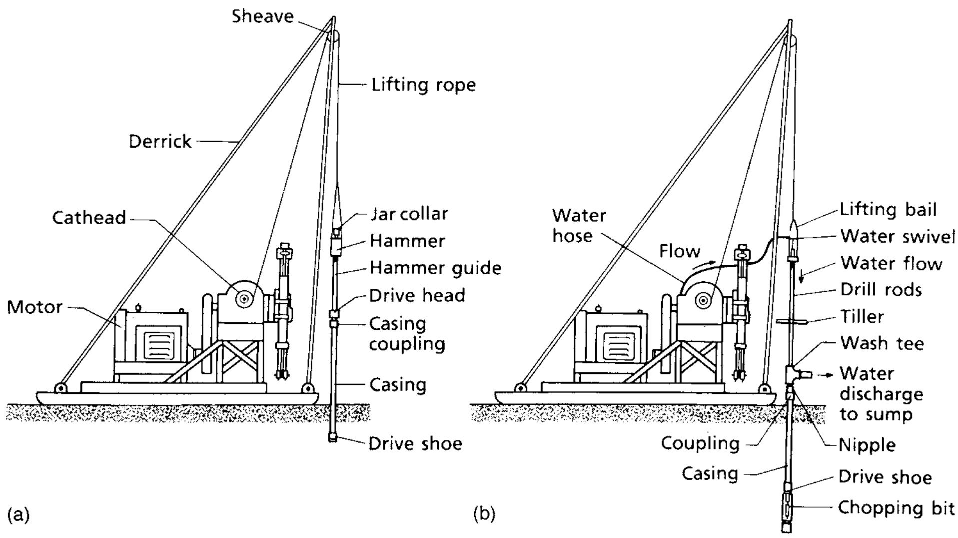


Figure 7.3

Wash-boring rig. (a) Driving the casing, (b) Advancing the hole.

United States, it rarely has been employed in Britain. This is mainly because wash boring does not lend itself to many of the ground conditions encountered and also to the difficulty of identifying strata with certainty.

Sampling in Soils

As far as soils are concerned, samples may be divided into two types, disturbed and undisturbed. Disturbed samples can be obtained by hand, by auger or from the clay cutter or shell of a boring rig. Samples of fine soils should be approximately 0.5 kg in weight, providing a sufficient size for index testing. The samples are sealed in jars. A larger sample is necessary if the particle size distribution of coarse soil is required, and this may be retained in a tough plastic sack. Care must be exercised when obtaining such samples to avoid loss of fines.

An undisturbed sample can be regarded as one that is removed from its natural condition without disturbing its structure, density, porosity, moisture content and stress condition. Although it must be admitted that no sample is ever totally undisturbed, every attempt must be made to preserve the original condition of such samples. Unfortunately, mechanical disturbances produced when a sampler is driven into the ground distort the soil structure. Furthermore, a change of stress condition occurs when a borehole is excavated.

Undisturbed samples may be obtained by hand from surface exposures, pits and trenches. Careful hand trimming is used to produce a regular block, normally a cube of about 250 mm dimensions. Block samples are covered with muslin and sealed with wax. Such samples are particularly useful when it is necessary to test specific horizons, such as shear zones.

As far as any undisturbed sampling tool is concerned, its fundamental requirement is that on being forced into the ground it should cause as little remoulding and displacement of the soil as possible. The amount of displacement is influenced by a number of factors. Firstly, there is the cutting shoe or edge of the sampler. A thin cutting shoe and sampling tube minimize displacement but they can be damaged easily, and they cannot be used in gravels and hard soils. Secondly, the internal diameter of the cutting shoe, D_i, should be slightly less than that of the sample tube, thereby providing inside clearance that reduces drag effects due to friction. Thirdly, the outside diameter of the cutting shoe, D_o, should be from 1 to 3% larger than that of the sampler, again to allow for clearance. The relative displacement of a sampler can be expressed by the area ratio, A_r:

$$A_r = \frac{D_i^2 - D_o^2}{D_o^2} \times \mathbf{100} \tag{7.1}$$

This ratio should be kept as low as possible, for example, displacement is minimized by keeping the area ratio below 15%. It should not exceed 25% (Hvorslev, 1949). Fourthly, friction also can be reduced if the tube has a smooth inner wall. A coating of light oil also may prove useful in this respect.

The standard sampling tube for obtaining samples from cohesive soils is referred to as the U100, having a diameter of 100 mm, a length of approximately 450 mm and walls 1.2 mm thick (Fig. 7.4). The cutting shoe should meet the requirements noted above. The upper end of the tube is fastened to a check valve that allows air or water to escape during driving and helps to hold the sample in place when it is being withdrawn. On withdrawal from the borehole, the sample is sealed in the tube with paraffin wax and the end caps screwed on. In soft materials, two or three tubes may be screwed together to reduce disturbance of the sample.

The standard type of sampler is suitable for clays with a shear strength exceeding 50 kPa. However, a thin-walled piston sampler should be used for obtaining clays with lower shear strength since soft clays tend to expand into the sample tube. Expansion is reduced by a piston in the sampler, the thin-walled tube being jacked down over a stationary internal piston, which, when sampling is complete, is locked in place and the whole assembly then is pulled (Fig. 7.5). Piston samplers range in diameter from 54 to 250 mm. A vacuum tends to be created between the piston and the soil sample, which thereby helps to hold it in place.

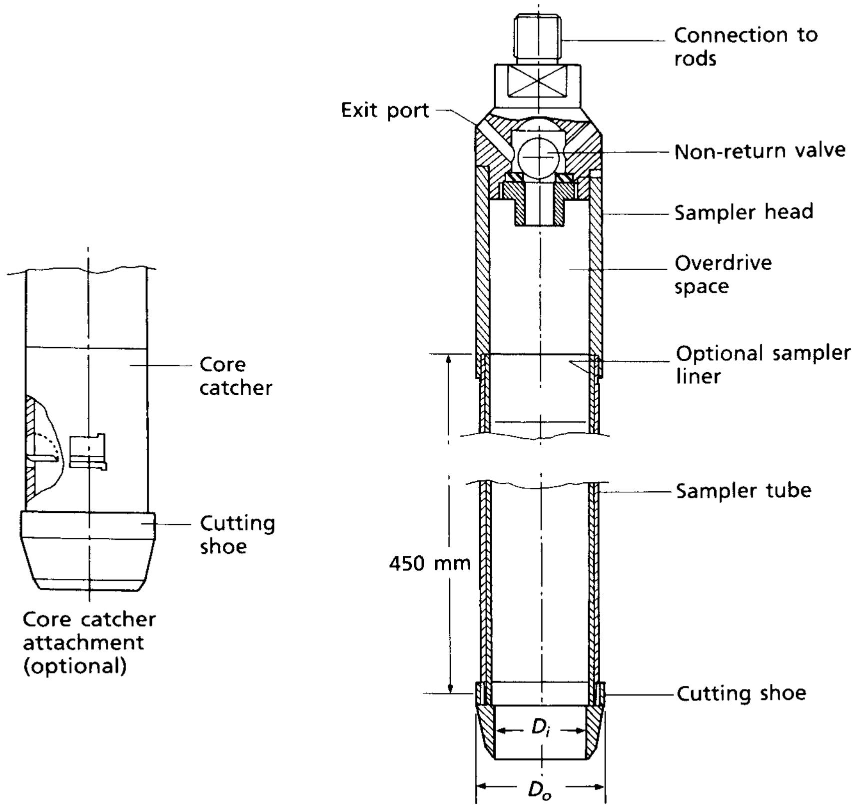


Figure 7.4

The general-purpose open-tube sampler, U100.

When continuous samples are required, particularly from rapidly varying or sensitive soils, a Delft sampler may be used (Fig. 7.6). This can obtain a continuous sample from ground level to depths of about 20 m. The core is retained in a self-vulcanising sleeve as the sampler is continuously advanced into the soil.

Sub-surface Exploration in Rocks and Sampling

Rotary drills are either skid-mounted, trailer mounted or, in the case of larger types, mounted on lorries (Fig. 7.7). They are used for drilling through rock, although they can penetrate and take samples from soil.

Rotary percussion drills are designed for rapid drilling in rock (Fig. 7.8). The rock is subjected to rapid high-speed impacts while the bit rotates, which brings about compression and shear

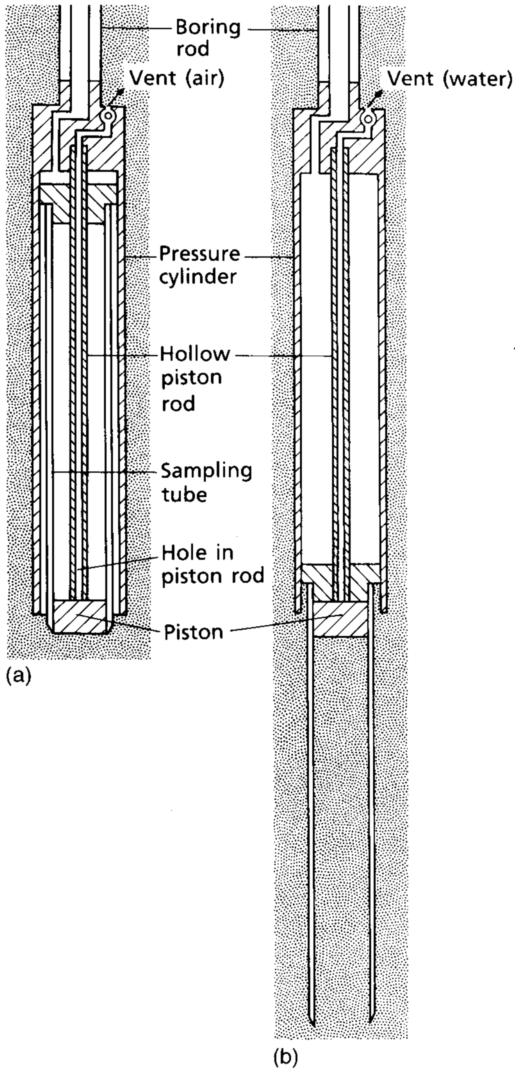


Figure 7.5

Piston sampler of hydraulically operated type. (a) Lowered to bottom of borehole, boring rod clamped in fixed position at the ground surface, (b) Sampling tube after being forced into the soil through water supplied through boring rod.

in the rock. Full-face bits, which produce an open hole, are used. These are usually of the studded, cruciform or tricone roller bit type (Fig. 7.9). The technique is most effective in brittle materials since it relies on chipping the rock. The rate at which drilling proceeds depends upon the type of rock, particularly on its strength, hardness and fracture index; the type of drill and drill bit; the flushing medium and the pressures used; as well as the experience of the drilling crew. Compressed air, water or mud may be used as the flush. If the drilling operation is standardized,

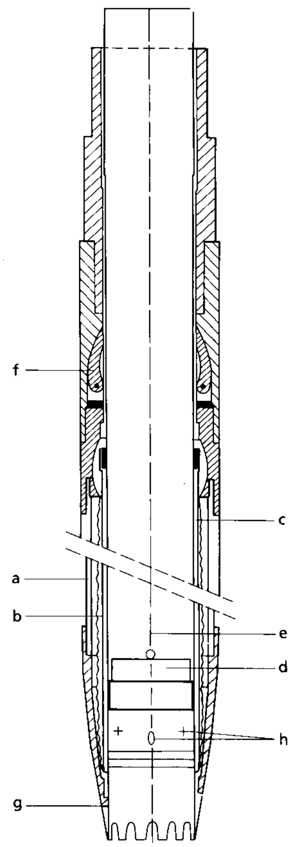


Figure 7.6

Section through a 66-mm continuous sampling apparatus. (a) Outer tube, (b) stocking tube over which pre-coated nylon stocking is slid, (c) plastic inner tube, (d) cap at top of sample, (e) steel wire to fixed point at the ground surface (tension cable), (f) sample-retaining clamps, (g) cutting shoe, (h) holes for entry of lubricating fluid.

then differences in the rate of penetration reflect differences in rock types. Drill flushings should be sampled at regular intervals, at changes in the physical appearance of the flushings and at significant changes in penetration rates. Interpretation of rotary percussion drillholes should be related to a cored drillhole near by. Rotary percussion drilling sometimes is used as a means of advancing a hole at low cost and high speed between intervals where core drilling is required.

For many engineering purposes, a solid, and as near as possible continuous rock core, is required for examination. The core is cut with a bit and housed in a core barrel (Fig. 7.10). The bit is set with diamonds or tungsten carbide inserts. In set bits, diamonds are set on the face of the matrix. The coarser surface set diamond and tungsten carbide tipped bits are used in softer formations. These bits generally are used with air rather than with water flush. Impregnated bits possess a matrix impregnated with diamond dust, and their grinding action

Figure 7.7

Medium-size, skid-mounted rotary drill.

is suitable for hard and broken formations. In fact, most core drilling is carried out using diamond bits, the type of bit used being governed by the rock type to be drilled. In other words, the harder the rock, the smaller the size and the higher the quality of the diamonds that are required in the bit. Tungsten bits are not suitable for drilling in very hard rocks. Thick-walled bits are more robust but penetrate more slowly than thin-walled bits. The latter produce a larger core for a given hole size. This is important where several reductions in size have to be made. Core bits vary in size, and accordingly core sticks range between 17.5 and 165 mm diameter. Other factors apart, generally the larger the bit, the better is the core recovery.

A variety of core barrels is available for rock sampling. The simplest type of core barrel is the single tube, but because it is suitable only for hard massive rocks, it rarely is used. In the single-tube barrel, the barrel rotates the bit and the flush washes over the core. In double-tube barrels, the flush passes between the inner and outer tubes. Double tubes may be of the rigid or swivel type. The disadvantage of the rigid barrel is that both the inner and outer tubes rotate together, and in soft rock this can break the core as it enters the inner tube.

Figure 7.8

Rotary percussion drilling rig.

It therefore is only suitable for hard rock formations. In the double-tube swivel-core barrel, the outer tube rotates while the inner tube remains stationary (Fig. 7.11). It is suitable for use in medium and hard rocks, and gives improved core recovery in soft friable rocks. The face-ejection barrel is a variety of the double-tube swivel-type in which the flushing fluid does not affect the end of the core. This type of barrel is a minimum requirement for coring badly shattered, weathered and soft rock formations. Triple-tube barrels are used for obtaining cores from very soft rocks and

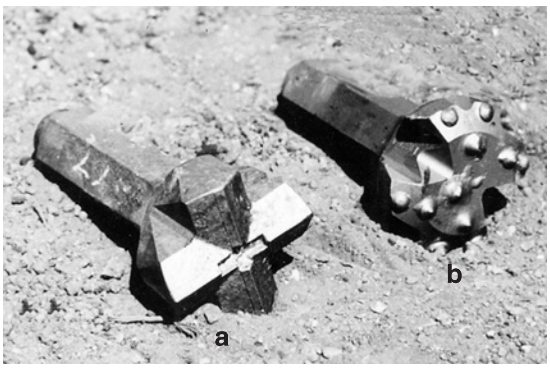


Figure 7.9

Full-face bits for rotary percussion, (a) cross-chisel or cruciform bit, (b) studded bit.

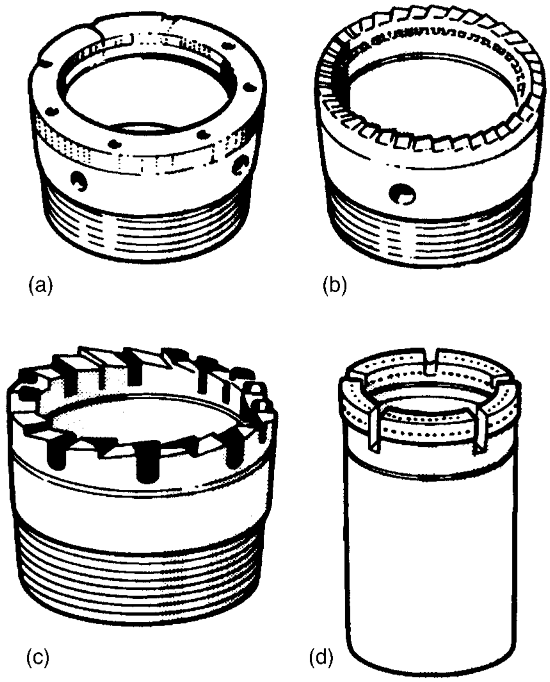


Figure 7.10

Some common types of coring bits, (a) surface-set diamond bit (bottom discharge), (b) stepped saw-tooth bit, (c) tungsten carbide bit, (d) impregnated diamond bit.

from highly jointed and cleaved rocks. This type of core barrel has an inner triple tube that is split into two halves longitudinally. Hence, when withdrawn from the outer casing of the core barrel the core can be observed and described without the risk of disturbance.

Both the bit and core barrel are attached by rods to the drill by which they are rotated. Either water or air is used as a flush. This is pumped through the drill rods and discharged at the bit. The flushing agent serves to cool the bit and to remove the cuttings from the drillhole. Bentonite is sometimes added to the water flush. It eases the running and pulling of casing by lubrication, it holds chippings in suspension and promotes drillhole stability by increasing flush returns through the formation of a filter skin on the walls of the hole.

Disturbance of the core is likely to occur when it is removed from the core barrel. Most rock cores should be removed by hydraulic extruders while the tube is held horizontal. To reduce disturbance during extrusion the inner tube of double-core barrels can be lined with a plastic sleeve before drilling commences. On completion of the core run, the plastic sleeve containing the core is withdrawn from the barrel.

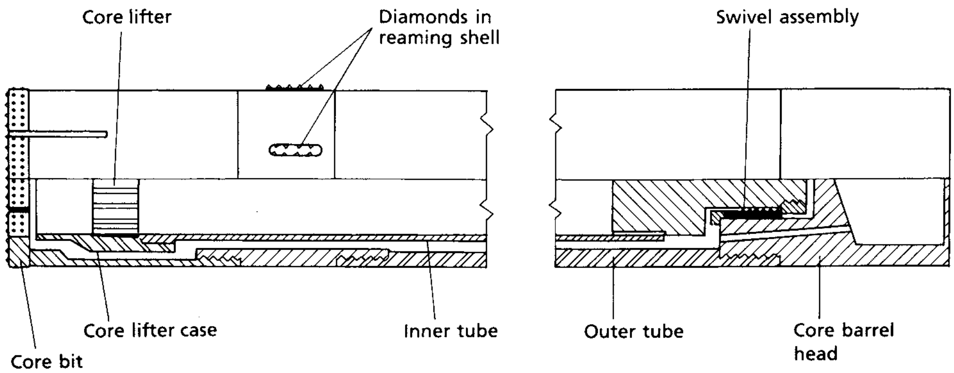


Figure 7.11

Double-tube swivel-type core barrel.

If casing is used for drilling operations, then it is drilled into the ground using a tungsten-carbide- or diamond-tipped casing shoe with air, water or mud flush. The casing may be inserted down a hole drilled to a larger diameter to act as conductor casing when reducing and drilling ahead in a smaller diameter, or it may be drilled or reamed in a larger diameter than the initial hole to allow continued drilling in the same diameter.

Many machines will core drill at any angle between vertical and horizontal. Unfortunately, inclined drillholes tend to go off line, the problem being magnified in highly jointed formations. In deeper drilling, the sag of the rods causes the hole to deviate. Drillhole deviation can be measured by an inclinometer.

The weakest strata generally are of the greatest interest, but these are the materials that are most difficult to obtain and most likely to deteriorate after extraction. Shales and mudstones are particularly prone to deterioration, and some may disintegrate completely if allowed to dry. Deterioration of suspect rock may be reduced by wrapping core material with aluminium foil or plastic sheeting. Core material may be photographed before it is removed from site. Zones of core loss or no recovery must be recorded as these could represent problem zones. Hawkins (1986) introduced the concept of lithology quality designation, LQD, which he defined as the percentage of solid core present greater than 100 mm in length within any lithological unit. He also recommended that the total core recovery, TCR, and the maximum intact core length, MICL, should be recorded.

A simple but nonetheless important factor is labelling of core material. This must record the site, the drillhole number and the position in the drillhole from which material was obtained. The labels themselves must be durable and properly secured. When rock samples are stored in a core box, the depth of the top and bottom of the core contained and of the separate core runs should be noted both outside and inside the box. Zones of core loss should be identified.

In Situ Testing

There are two categories of penetrometer tests, the dynamic and the static. Both methods measure the resistance to penetration of a conical point offered by the soil at any particular depth. Penetration of the cone creates a complex shear failure and thus provides an indirect measure of the in situ shear strength of the soil.

The most widely used dynamic method is the standard penetration test. This empirical test, which was designed initially for use in sands, consists of driving a split-spoon sampler, with an outside diameter of 50 mm, into the soil at the base of a borehole. If the test is carried out in gravelly soils, then the cutting shoe is replaced by a 60° cone. Drivage is accomplished by a trip hammer, weighing 65 kg, falling freely through a distance of 760 mm onto the drive head, which is fitted at the top of the rods (Fig. 7.12). First, the split-spoon is driven 150 mm into the soil at the bottom of the borehole. It then is driven a further 300 mm, and the number of blows required to drive this distance is recorded. The blow count is referred to as the N value from which the relative density of coarse soil can be assessed (Table 7.3). Refusal is regarded as 50 blows. In deep boreholes, the standard penetration test suffers the disadvantage that the load is applied at the top of the rods so that some of the energy from the blow is dissipated in the rods. Hence, with increasing depth the test results become more suspect.

The results obtained from the standard penetration test provide an evaluation of the degree of compaction of sands, and the *N* values may be related to the values of the angle of internal friction, ϕ, and the allowable bearing capacity. The lowest values of the angle of internal friction given in Table 7.3 are conservative estimates for uniform clean sand, and they should be reduced by at least 5° for clayey sand. The upper values apply to well graded sand and may be increased by 5° for gravelly sand. Terzaghi and Peck (1968) suggested that the relative density for very fine or silty submerged sand with a standard penetration value *N*' greater than 15 would be nearly equal to that of a dry sand with a standard penetration value, *N*, where:

$$N = \mathbf{15} + \frac{\mathbf{1}}{\mathbf{2}}(N' - \mathbf{15}) \tag{7.2}$$

If this correction was not made, Terzaghi and Peck suggested that the relative density of even moderately dense very fine or silty submerged sand might be overestimated by the results of standard penetration tests. In gravel deposits, care must be taken to determine whether a large gravel size may have influenced the results. Usually, in the case of

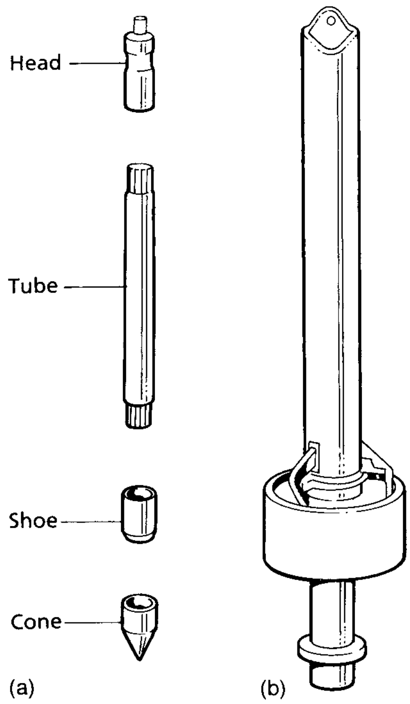


Figure 7.12

Standard penetration test equipment, (a) split spoon sampler with shoe or cone end caps, (B) trip hammer.

gravel, only the lowest values of *N* are taken into account. The standard penetration test also can be employed in stiff clays, weak rocks and in the weathered zones of harder rocks (Table 7.3).

The most widely used static method employs the Dutch cone penetrometer (Fig. 7.13). It is particularly useful in soft clays and loose sands, where boring operations tend to disturb in situ values. In this technique, a tube and inner rod with a conical point at the base are hydraulically advanced into the ground, the reaction being obtained from pickets screwed into the ground. The cone has a cross-sectional area of 1000 mm^2 with an angle of 60°. At approximately every 300 mm depth, the cone is advanced ahead of the tube a distance of 50 mm and the maximum resistance noted. The tube then is advanced to join the cone after each measurement and the process repeated. The resistances are plotted against their corresponding depths so as to give a profile of the variation in consistency (Fig. 7.14). One type of Dutch cone penetrometer has a sleeve behind the cone that can measure side friction. The ratio of sleeve resistance to that of cone resistance is higher in fine than in coarse soils, thus affording a tentative estimate of the type of soil involved.

Table 7.3. Relative density and consistency of soil

(a) Relative density of sand and SPT values, and relationship to angle of friction

SPT(N)	Relative density (D_r)	Description of compactness	Angle of internal friction (φ)
4	0.2	Very loose	Under 30°
4–10	0.2–0.4	Loose	30–35°
10–30	0.4–0.6	Medium dense	35–40°
30–50	0.6–0.8	Dense	40–45°
Over 50	0.8–1.0	Very dense	Over 45°

(b) N values, consistency and unconfined compressive strength of cohesive soils

N	Consistency	Unconfined compressive strength (kPa)
Under 2	Very soft	Under 20
2–4	Soft	20–40
5–8	Firm	40–75
9–15	Stiff	75–150
16–30	Very stiff	150–300
Over 30	Hard	Over 300

In the piezocone, a cone penetrometer is combined with a piezometer, the latter being located between the cone and the friction sleeve. The pore water pressure is measured by the piezometer at the same time as the cone resistance, and sleeve friction is recorded. Because of the limited thickness of the piezometer (the filter is around 5 mm), much thinner layers can be determined with greater accuracy than with a conventional cone penetrometer. If the piezocone is kept at a given depth so that the pore water pressure can dissipate with time, then this allows assessment of the in situ permeability and consolidation characteristics of the soil to be made (Sills and Hird, 2005).

Because soft clays may suffer disturbance when sampled and therefore give unreliable results when tested for strength in the laboratory, a vane test may be used to measure the in situ undrained shear strength. Vane tests can be used in clays that have a consistency varying from very soft to firm. In its simplest form, the shear vane apparatus consists of four blades arranged in cruciform fashion and attached to the end of a rod (Fig. 7.15). To eliminate the effects of friction of the soil on the vane rods during the test, all rotating parts, other than the vane, are enclosed in guide tubes. The vane normally is housed in a protective shoe. The vane and rods are pushed into the soil from the surface or the base of a borehole to a point 0.5 m above the required depth of testing. Then, the vane is pushed out of the protective shoe and advanced to the test position. It then is rotated at a rate of 6 to 12° per minute. The torque is applied to the vane rods by means of a torque-measuring instrument mounted at ground

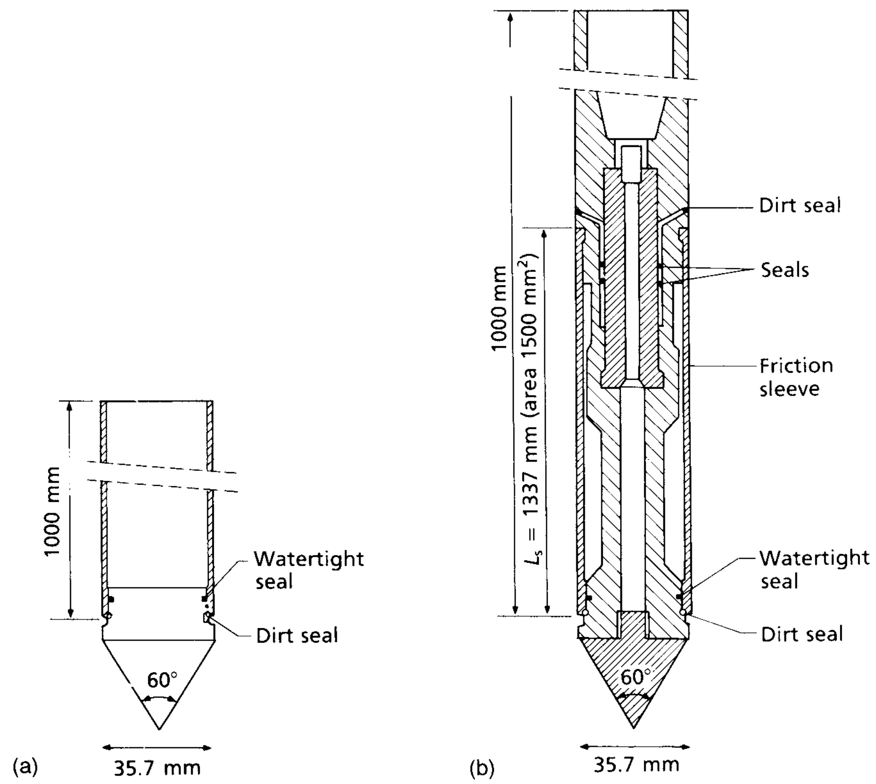


Figure 7.13

An electric penetrometer tip. (a) without friction sleeve, (b) with friction sleeve.

level and clamped to the borehole casing or rigidly fixed to the ground. The maximum torque required for rotation is recorded. When the vane is rotated, the soil fails along a cylindrical surface defined by the edges of the vane as well as along the horizontal surfaces at the top and bottom of the blades. The shearing resistance is obtained from the following expression:

$$\tau = \frac{M}{\pi\left(\frac{D^2H}{2} + \frac{D^3}{6}\right)} \tag{7.3}$$

where τ is the shearing resistance, D and H are the diameter and height of the vane, respectively, and M is the torque. Tests in clays with high organic contents or with pockets of sand or silt are likely to produce erratic results. The results therefore should be related to borehole evidence.

Loading tests can be carried out on loading plates in soils or rocks (Fig. 7.16a). However, just because the ground immediately beneath a plate is capable of carrying a heavy load without excessive settlement, this does not necessarily mean that the ground will carry the proposed

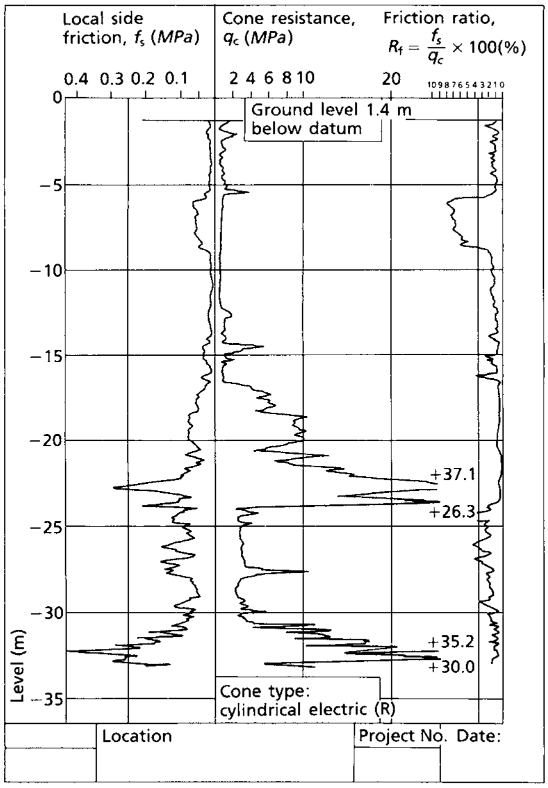


Figure 7.14

Typical record of cone penetrometer test.

structural load. This is especially the case where a weaker horizon occurs at depth but is still within the influence of the bulb of pressure that will be generated by the structure (Fig. 7.16b). The plate-loading test provides information by which the bearing capacity and settlement characteristics of a foundation can be assessed (Matthews and Clayton, 2004). Such a test may be carried out in a trial pit, usually at excavation base level. Plates vary in size from 0.15 to 1.0 m in diameter. Tomlinson (2001) recommended that a 300 mm plate was the minimum size that should be used in stiff fissured clays in order to obtain their undrained shear strength. If the deformation modulus is required for such soils, then Tomlinson recommended a plate size of 750 mm. The plate should be bedded properly and the test carried out on undisturbed material so that reliable results may be obtained. The load is applied by a hydraulic jack bearing against beams supporting kentledge, or reaction may be provided by ground anchors or tension piles installed on either side of the load position. The load may be applied in increments, either of one-fifth of the proposed bearing pressure or in steps of 25 to 50 kPa (these are smaller in soft soils, i.e. where the settlement under the first increment of 25 kPa is greater than 0.002D, D being the diameter of the plate). Successive increments

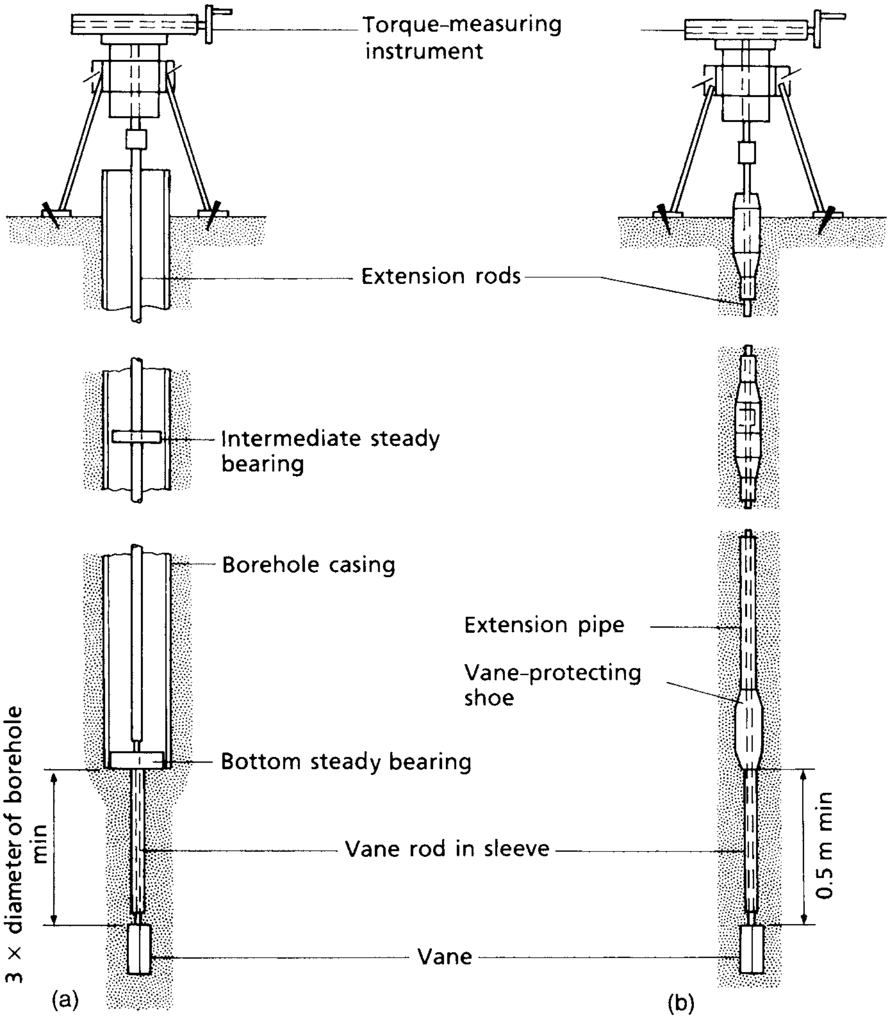


Figure 7.15

Shear vane tests, (a) borehole vane test, (b) penetration vane test.

should be made after settlement has ceased. The test generally is continued up to two or three times the proposed loading or in clays until settlement equal to 10 to 20% of the plate dimension is reached or the rate of increase of settlement becomes excessive. When the final increment is applied in clays, the load should be maintained until the rate of settlement becomes less than 0.1 mm in 2 h. This can be regarded as the completion of the primary consolidation stage. Settlement curves can be drawn with this information from which the ultimate loading can be determined and an evaluation of Young's modulus made. At the end of the consolidation stage, the plate can be unloaded in the same incremental steps in order to obtain an unloading curve.

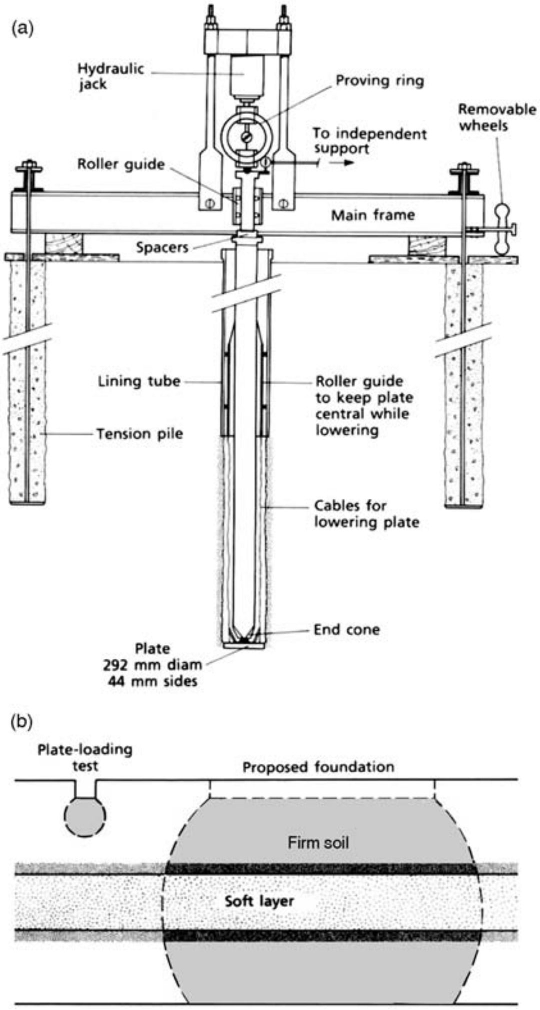


Figure 7.16

(a) Plate load test. With the permission of the Director of the Building Research Establishment. (b) Bulb of pressure developed beneath a foundation compared with one developed beneath a plate load test.

The screw plate is a variant of the plate load test in which a helical screw is rotated into the ground to the depths at which the test is to be conducted. The test has the advantage that no excavation or drilling are needed, and it can be performed beneath the water table. Unfortunately, however, screwing the plate into the soil may cause disturbance around the plate.

Large-plate-bearing tests frequently are used to determine the value of Young's modulus of the foundation rock at large civil engineering sites, such as at dam sites. Loading of the order of several mega-newtons is required to obtain measurable deformation of representative areas. The area of rock load is usually 1 m^2. Tests may be carried out in specially excavated galleries in order to provide a sufficiently strong reaction point for the loading jacks to bear against. The test programme normally includes cycles of loading and unloading. Such tests show that during loading a noticeable increase in rigidity normally occurs in the rock mass and that during unloading a very small deformation occurs for the high stresses applied, with very large recuperation of deformation being observed for stresses near zero. This is due to joint closure. Once the joints are closed, the adhesion between the faces prevents their opening until a certain unloading is reached. However, when brittle rocks such as granite, basalt and limestone have been tested, they generally have given linear stress–strain curves and have not exhibited hysteresis.

Variations of the plate-load test include the freyssinet jack test. This is placed in a narrow slit in a rock mass and then grouted into position so that each face is in uniform contact with the rock. Pressure then is applied to the jack. Unless careful excavation, particularly if blasting, takes place in

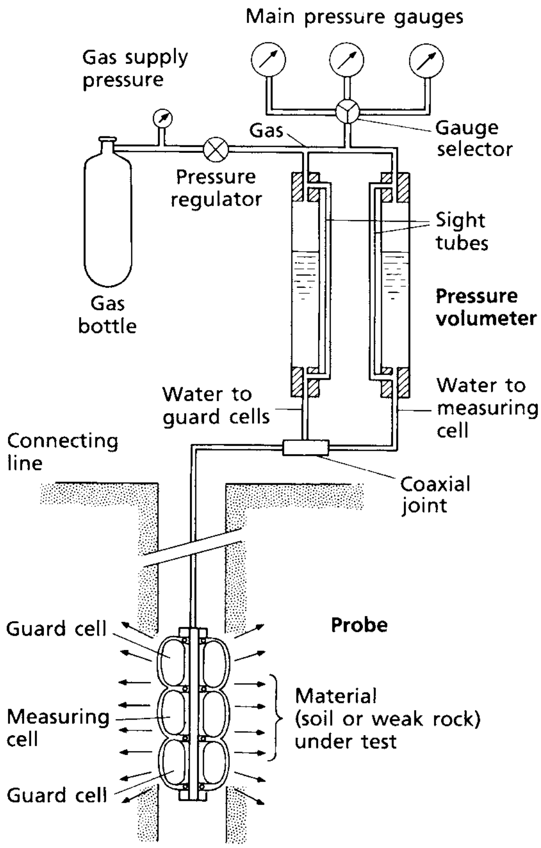


Figure 7.17

The Menard pressuremeter.

the testing area, the results of a test may be worthless. All loose rock must be removed before cutting the slot. Freyssinet jacks can be used to measure residual stress as well as Young's modulus.

The Menard pressuremeter is used to determine the in situ strength of the ground (Fig. 7.17). It is particularly useful in those soils from which undisturbed samples cannot be obtained readily. This pressuremeter consists essentially of a probe that is placed in a borehole at the appropriate depth and then expanded. Where possible the test is carried out in an unlined hole but if necessary a special slotted casing is used to provide support. The probe consists of a cylindrical metal body over which are fitted three cylinders. A rubber membrane covers the cylinders and is clamped between them to give three independent cells. The cells are inflated with water and a common gas pressure is applied by a volumeter located at the surface, thus a radial stress is applied to the soil. The deformations produced by the central cell are indicated on the volumeter. A simple pressuremeter test consists of 10 or more equal pressure increments with corresponding volume change readings, taken to the ultimate failure strength of the soil concerned. Four volume readings are made at each pressure step at time intervals of 15, 30, 60 and 120 s after the pressure has stabilized. It is customary to unload the soil at the end of the elastic phase of expansion and to repeat the test before proceeding to the ultimate failure pressure. This test thus provides the ultimate bearing capacity of soils as well as their deformation modulus. The test can be applied to any type of soil, and takes into account the influence of discontinuities. It also can be used in weathered zones of rock masses and in weak rocks such as some shales and marls. It provides an almost continuous method of in situ testing.

The major advantage of a self-boring pressuremeter is that a borehole is unnecessary. Consequently, the interaction between the probe and the soil is improved. Self-boring is brought about either by jetting or using a chopping tool (Fig. 7.18). For example, the camkometer has a special cutting head so that it penetrates soft ground to form a cylindrical cavity of its exact dimensions and thereby creates a minimum of disturbance (Fig. 7.18b). The camkometer measures the lateral stress, undrained stress–strain properties and the peak stress of soft clays and sands in situ. Clarke (1990) described the use of the self-boring pressuremeter test to determine the in situ consolidation characteristics of clay soils.

A dilatometer can be used in a drillhole to obtain data relating to the deformability of a rock mass (Fig. 7.19). These instruments range up to about 300 mm in diameter and over 1 m in length and can exert pressures of up to 20 MPa on the drillhole walls. Diamentral strains can be measured either directly along two perpendicular diameters or by measuring the amount of liquid pumped into the instrument. The last method is less accurate and is only used when the rock is very deformable.

In an in situ shear test, a block of rock is sheared from a rock surface while a horizontal jack exerts a vertical load. It is advantageous to make the tests inside galleries, where

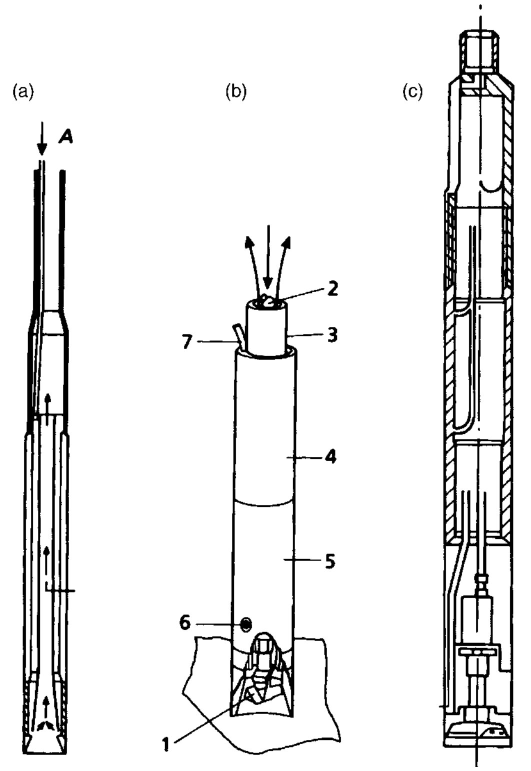


Figure 7.18

(a) Schematic diagram of self-boring pressuremeter. This pressuremeter is available in three diameters (65, 100 and 132 mm). The injection fluid flow that can be applied is limited by the section of flexible pipe (A). Consequently, this pressuremeter can be used only in fine sands or soft cohesive soils. (b) The Camkometer breaks up the soil with a chopping tool rotated by a drill string driven from the surface. The chopping tool (1) is driven by a hollow middle rod (2) through which water is driven under pressure. This rod turns freely within a tube (3) used for removing sediment to the surface. There also is a tube (4) that carries the pressiometric cell (5) that may be equipped with a pore pressure tap (6). The pressiometeric cell supply and measurement lines (7) run through the annulus between the two tubes. (c) Self-boring pressuremeter probe with built in motor.

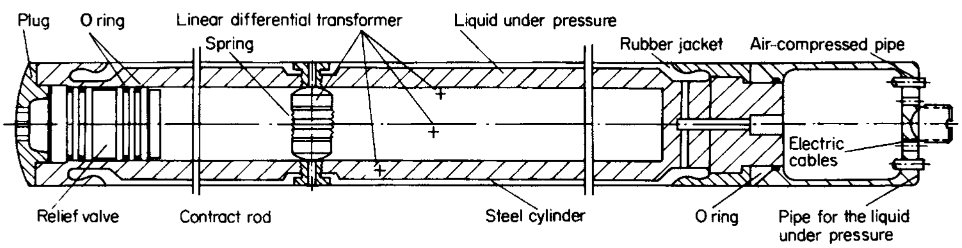


Figure 7.19

A dilatometer.

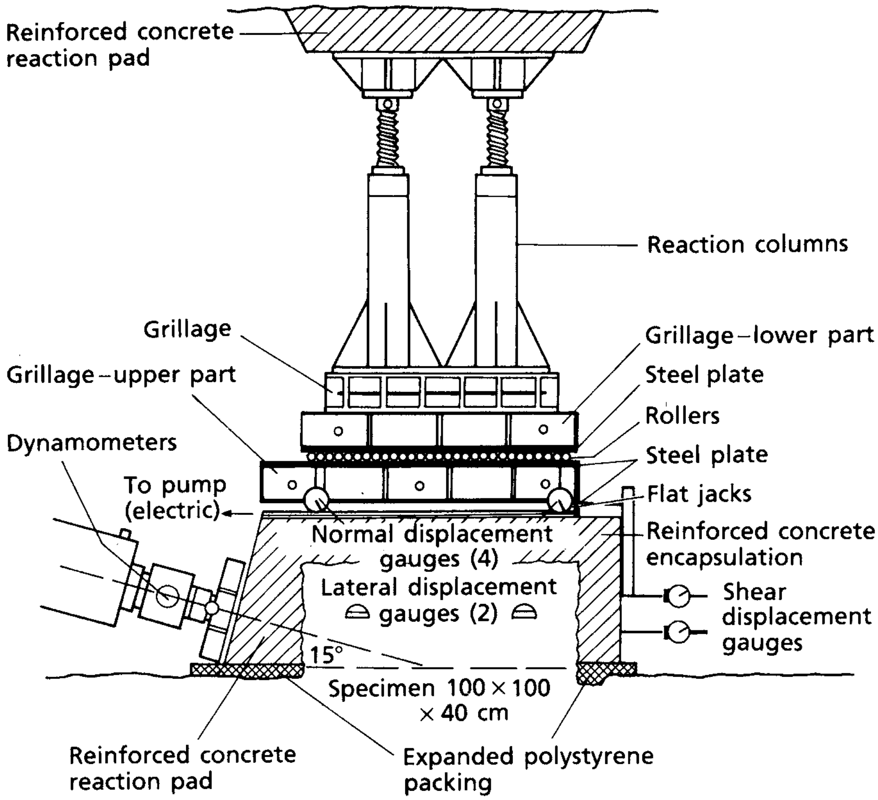


Figure 7.20

The in situ shear test apparatus.

reactions for the jacks are readily available (Fig. 7.20). The tests are performed at various normal loads and give an estimate of the angle of shearing resistance and cohesion of the rock. The value of this test in very jointed and heterogeneous rocks is severely limited both because of the difficulty in isolating undisturbed test blocks and because the results cannot be translated to the scale of conditions of the actual structure and its foundation. In situ shear tests usually are performed on blocks, 700 × 700 mm^2, cut in the rock. These tests can be made on the same rock where it shows different degrees of alteration and can be used to derive the shear strength parameters along discontinuities.

Field Instrumentation

When some degree of risk is involved in construction, some type of field instrumentation may be required in order to provide a continual check on the stability of the structure during its life span. Furthermore, field observations of both the magnitude and rate of subsurface ground movements are needed in connection with deep excavations, slope stability and earth and rockfill dam construction. Such instrumentation needs to assess the pore water pressure, deformation and stress and strain in the ground. However, an instrumentation programme does not usually constitute part of a site investigation.

Surface deformation either in the form of settlements or horizontal movements can be monitored by precise surveying methods, the use of electronic distance measurement, EDM, or laser equipment providing particularly accurate results. Settlement, for example, can be recorded by positioning reference marks on the structures concerned and readings being taken by precise surveying methods. The observations are related to nearby bench marks. Vertical movements also can be determined by settlement tubes or by water-level or mercury-filled gauges.

Borehole extensometers are used to measure the vertical displacement of the ground at different depths. A single-rod extensometer is anchored in a borehole, and movement between the anchor and the reference tube is monitored. Multiple-rod installations monitor displacements at various depths using rods of various lengths. Each rod is isolated by a close-fitting plastic sleeve and the complete assembly is grouted into place, fixing the anchors to the ground while allowing free movement of each rod within its sleeve (Fig. 7.21a). A precise borehole extensometer consists of circular magnets embedded in the ground, which act as markers, and reed switch sensors move in a central access tube to locate the positions of the magnets (Fig. 7.21b).

An inclinometer is used to measure horizontal movements below ground and relies on the measurement of the angle a pendulum makes with the vertical at given positions in a specially cased borehole (Fig. 7.22). The gravity-operated pendulum transmits electrical signals to a recorder, and a vertical profile thereby is obtained. Sets of readings over a period of time enable both the magnitude and rate of horizontal movement to be determined.

The measurement of stress, contact pressures and stress change may be made in two ways. Strain may be measured and then converted to stress, or stress may be measured directly by an earth pressure cell such as the Glotzl cell. This is a hydraulic (flat diaphragm) cell that has a high stiffness at constant temperature and is used for measuring contact pressures. An earth cell must be placed in position in such a way as to minimize disturbance of the stress and strain distribution. Ideally, an earth pressure cell should have the same elastic properties as the surrounding soil. This, of course, cannot be attained, and in order to minimize the magnitude of error (i.e. the cell action factor), the ratio of the thickness to the diameter of the cell should not exceed 0.2 and the ratio of the diameter to the deflection of the diaphragm must be 2000 or greater. Most strain measurements in soils and soft rocks are deformation measurements that are interpreted in terms of strain. A strain cell is required to move with the soil without causing it to be reinforced. In order to record strain, it is necessary to monitor the relative movements of two fixed points at either end of a gauge length. Strain cells with a

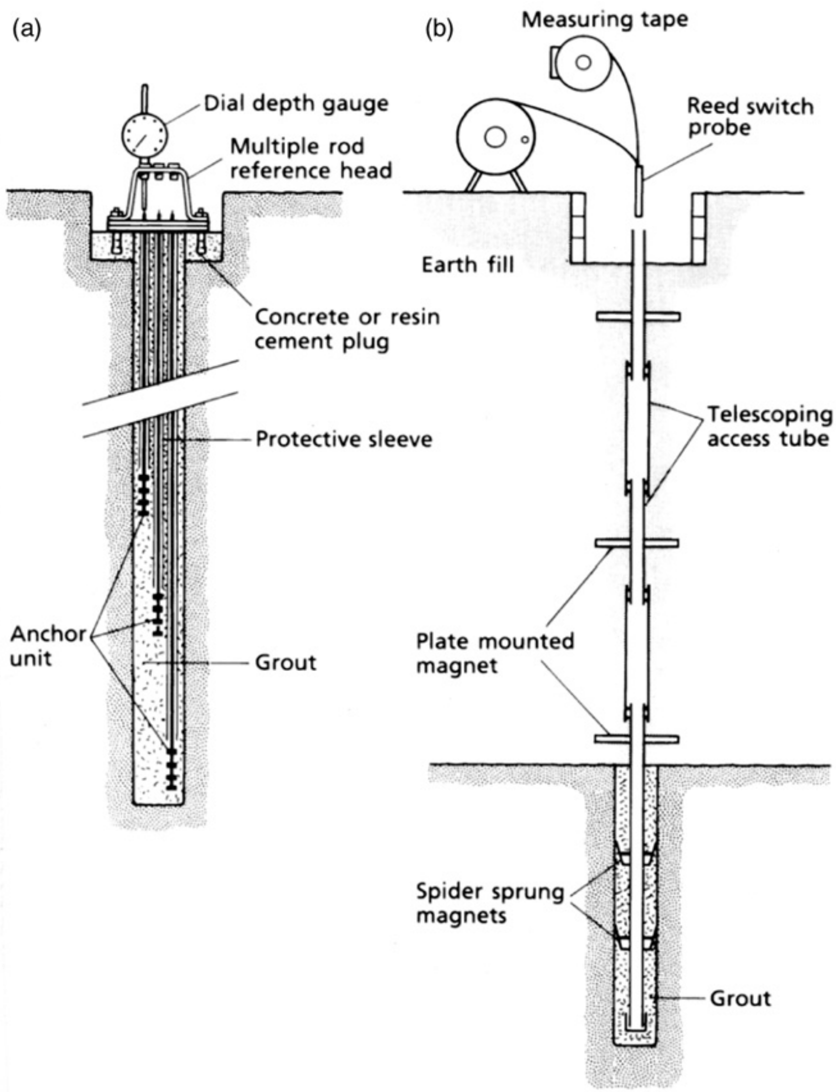


Figure 7.21

(a) Multiple rod extensometer. (b) Magnetic probe extensometer.

positive connection between their end plates have difficulty in measuring small strains. This can be overcome by substituting separated strain cells at either end of the gauge length.

Geophysical Methods: Indirect Site Exploration

A geophysical exploration may be included in a site investigation for an important engineering project in order to provide subsurface information over a large area at reasonable cost.

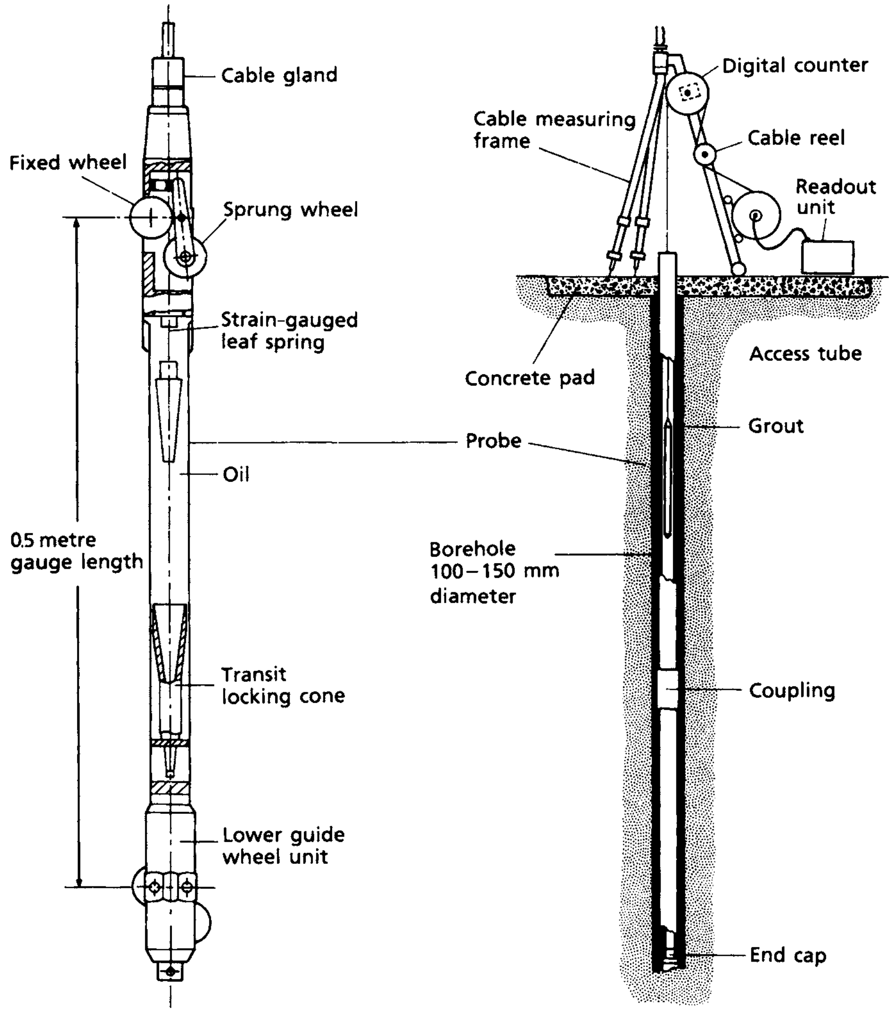


Figure 7.22

Borehole inclinometer.

The information obtained may help eliminate less favourable alternative sites, may aid the location of test holes in critical areas and may prevent unnecessary repetitive boring or drilling in fairly uniform ground. A geophysical survey not only helps to locate the position of boreholes or drillholes but also detects variations in subsurface conditions between them. Boreholes and drillholes provide information about the strata where they are sunk but provide no information about the ground in between. Nonetheless, boreholes or drillholes to aid interpretation and correlation of the geophysical measurements are an essential part of any geophysical survey. Therefore, an appropriate combination of direct and indirect methods often can yield a high standard of results.

Geophysical methods are used to determine the geological sequence and structure of subsurface rocks by the measurement of certain physical properties or forces. The properties that are made most use of in geophysical exploration are density, elasticity, electrical conductivity, magnetic susceptibility and gravitational attraction. In other words, seismic and resistivity methods record the artificial fields of force applied to the area under investigation, while magnetic and gravitational methods measure natural fields of force. The former techniques have the advantage over the latter in that the depth to which the forces are applied can be controlled. By contrast, the natural fields of force are fixed and can only be observed and not controlled. Seismic and resistivity methods are more applicable to the determination of horizontal or near horizontal changes or contacts, whereas magnetic and gravimetric methods generally are used to delineate lateral changes or vertical structures.

In a geophysical survey, measurements of the variations in certain physical properties usually are taken in a traverse across the surface, although they may be made in order to log a borehole. Generally speaking, observations should be close enough for correlation between them to be obvious, so enabling interpolation to be carried out without ambiguity (McDowell et al., 2002). Anomalies in the physical properties measured generally reflect anomalies in the geological conditions. The ease of recognizing and interpreting these anomalies depends on the contrast in physical properties that, in turn, influence the choice of method employed.

The actual choice of method to be used for a particular survey may not be difficult to make. The character and situation of the site also have to be taken into account, especially in built-up areas, which may be unsuitable for one or other of the geophysical methods either because of the presence of old buildings or services on the site, interference from some source or lack of space for carrying out the survey. When dealing with layered rocks, provided their geological structure is not too complex, seismic methods have a distinct advantage in that they give more detailed, precise and unambiguous information than any other method. On the other hand, electrical methods may be preferred for small-scale work, where the structures are simple. On occasions more than one method may be used to resolve the same problem. However, as McCann et al. (1997) pointed out, if there is any doubt about the feasibility of a geophysical survey, then a trial survey should be undertaken to determine the most suitable method. McCann et al. also noted that modern geophysical equipment allows downloading of the results to a suitable portable PC so that a preliminary interpretation can be carried out daily. In this way data can be checked, plotted and evaluated, and then can be compared with data gathered by other means. As a result, any errors can be recognized and, if necessary, traverses re-run to gather better information. Indeed, the survey programme can be modified in the light of any new information obtained.

Seismic Methods

The sudden release of energy from the detonation of an explosive charge in the ground or the mechanical pounding of the surface generates shock waves that radiate out in a hemispherical wave front from the point of release. The waves generated are compressional, P, dilational shear, S, and surface waves. The velocities of the shock waves generally increase with depth below the surface since the elastic moduli increase with depth. The compressional waves travel faster, and are generated and recorded more easily than shear waves. They are therefore used almost exclusively in seismic exploration. The shock wave velocity depends on many variables, including rock fabric, mineralogy and pore water. In general, velocities in crystalline rocks are high to very high (Table 7.4). Velocities in sedimentary rocks increase with amount of consolidation and decrease in pore fluids and with increase in the degree of cementation and diagenesis. Unconsolidated sedimentary deposits have maximum velocities varying as a function of the volume of voids, either air filled or water filled, mineralogy and grain size.

When seismic waves pass from one layer to another in the ground, some energy is reflected back towards the surface while the remainder is refracted. Thus, two methods of seismic surveying can be distinguished, that is, seismic reflection and seismic refraction. Measurement of the time taken from the generation of the shock waves until they are recorded by detector arrays forms the basis of the two methods.

The seismic reflection method is the most extensively used of all geophysical techniques, its principal employment being in the oil industry. In this technique, the depth of investigation is large compared with the distance from the shot to detector array. This is to exclude refraction waves. Indeed, the method is able to record information from a large number of horizons down to depths of several thousands of metres.

In seismic refraction, one ray approaches the interface between two rock types at a critical angle that means that, if the ray is passing from a low, V_o, to a high velocity V_1 layer, it will be refracted along the upper boundary of the latter layer (Fig. 7.23). After refraction, the pulse travels along the interface with velocity V_1. The material at the boundary is subjected to

Table 7.4. Velocities of compressional waves of some common rocks

	V_p (km s^{-1})		**V_p (km s^{-1})**
Igneous rocks		*Sedimentary rocks*	
Basalt	5.2–6.4	Gypsum	2.0–3.5
Dolerite	5.8–6.6	Limestone	2.8–7.0
Gabbro	6.5–6.7	Sandstone	1.4–4.4
Granite	5.5–6.1	Shale	2.1–4.4
Metamorphic rocks		*Unconsolidated deposits*	
Gneiss	3.7–7.0	Alluvium	0.3–0.6
Marble	3.7–6.9	Sands and gravels	0.3–1.8
Quartzite	5.6–6.1	Clay (wet)	1.5–2.0
Schist	3.5–5.7	Clay (sandy)	2.0–2.4

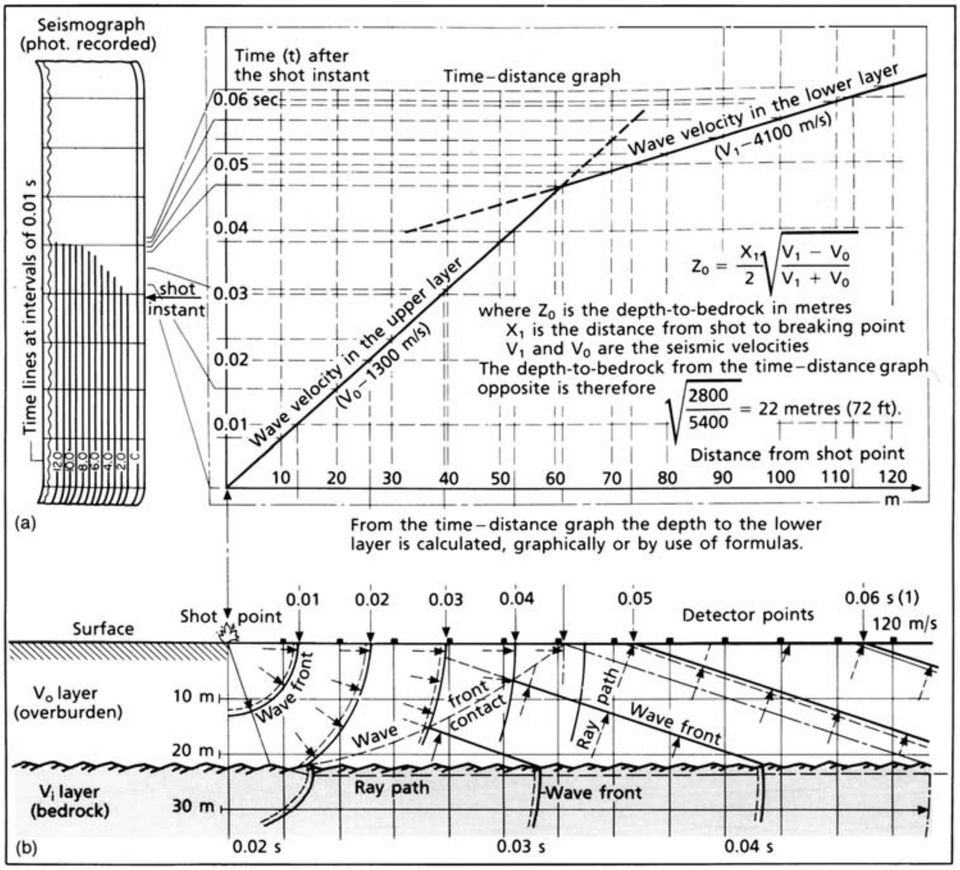


Figure 7.23

Time–distance graphs for a theoretical single-layer problem, with parallel interface. With non-parallel interfaces, both forward and reverse profiles must be surveyed.

oscillating stress from below. This generates new disturbances along the boundary that travel upwards through the low velocity rock and eventually reach the surface.

At short distances from the point where the shock waves are generated, the geophones record direct waves, while at a critical distance, both the direct and refracted waves arrive at the same time. Beyond this, because the rays refracted along the high velocity layer travel faster than those through the low velocity layer above, they reach the geophones first. In refraction work the object is to develop a time–distance graph that involves plotting arrival times against geophone spacing (Fig. 7.23). Hence, the distance between geophones, together with the total length and arrangement of the array, has to be chosen carefully to suit each particular problem.

The most common arrangement in refraction work is profile shooting. Here, the explosive shot or impact points and geophones are laid out in long lines, with geophones receiving refracted waves from the impacts or shots fired. The process is repeated at uniform intervals. In many surveys for civil engineering purposes where it is required to determine depth to bedrock, it may be sufficient to record from two shotpoint distances at each end of the receiving spread. By traversing in both directions in simple geological conditions, the angle of dip can be determined.

In the simple case of refraction by a single high velocity layer at depth, the travel time for the seismic wave that proceeds directly from the impact point to the detectors and the travel time for the critical refracted wave to arrive at the geophones are plotted graphically against geophone spacing (Fig. 7.23). The depth, Z, to the high velocity layer then can be obtained from the graph by using the expression:

$$Z = \frac{X}{2}\left(\frac{V_1 - V_0}{V_1 + V_0}\right) \tag{7.4}$$

where V_o is the speed in the low velocity layer, V_1 the speed in the high velocity layer and X is the critical distance. The method also works for multi-layered rock sequences if each layer is thick enough and transmits seismic waves at higher speeds than the one above it. However, in the refraction method, a low velocity layer underlying a high velocity layer usually cannot be detected as in such an inversion the pulse is refracted into the low velocity layer. Also, a layer of intermediate velocity between an underlying refractor and overlying layers can be masked as a first arrival on the travel-time curve. The latter is known as a blind zone. The position of faults also can be estimated from the time–distance graphs.

As noted in the preceding text, the velocity of shock waves is related to the elastic moduli and can therefore provide data relating to the engineering performance of the ground. Young's modulus, E, and Poisson's ratio, υ, can be derived by using the following expressions if the density, ρ, and compressional, V_p, and shear, V_s, wave velocities are known:

$$E = \rho V_p^2 \frac{(1+\upsilon)(1-2\upsilon)}{(1-\upsilon)} \tag{7.5}$$

or

$$E = 2V_S^2\rho(1+\upsilon) \tag{7.6}$$

or

$$E = \frac{V_s^2}{g}\rho\left[\frac{3(V_p/V_s)^2 - 4}{(V_p/V_s)^2 - 1}\right] \tag{7.7}$$

or

$$\upsilon = \frac{0.5\,(V_P/V_S)^2 - 1}{(V_P/V_S)^2 - 1} \tag{7.8}$$

where g is the acceleration due to gravity.

These dynamic moduli correspond to the initial tangent moduli of the stress–strain curve for an instantaneously applied load and are usually higher than those obtained in static tests. The frequency and nature of discontinuities within a rock mass affect its deformability. In other words, a highly discontinuous rock mass exhibits a lower compressional wave velocity than a massive rock mass of the same type. The influence of discontinuities on the deformability of a rock mass can be estimated from a comparison of its in situ compressional velocity, V_{pf}, and the laboratory sonic velocity, V_{pl}, determined from an intact specimen taken from the rock mass. The velocity ratio, V_{pf}/V_{pl}, reflects the deformability and so can be used as a quality index. A comparison of the velocity ratio with other rock quality indices is given in Table 2.7.

Resistivity Methods

The resistivity of rocks and soils varies within a wide range. Since most of the principal rock forming minerals are practically insulators, the resistivity of rocks and soils is determined by the amount of conducting mineral constituents and the content of mineralized water in the pores. The latter condition is by far the dominant factor, and in fact, most rocks and soils conduct an electric current only because they contain water. The widely differing resistivity values of the various types of impregnating water can cause variations in the resistivity of rocks ranging from a few tenths of an ohm-metre to hundreds of ohm-metres (Ω m) as can be seen from Table 7.5.

In the resistivity method, an electric current is introduced into the ground by means of two current electrodes and the potential difference between two potential electrodes is measured. It is preferable to measure the potential drop or apparent resistance directly in ohms rather than observe both current and voltage. The ohms value is converted to apparent resistivity by use of a factor that depends on the particular electrode configuration in use (see below).

Table 7.5. Resistivity of some types of natural water

Type of water	Resistivity in Ωm
Meteoric water, derived from precipitation	30 to 1000
Surface waters, in districts of igneous rocks	30 to 500
Surface waters, in districts of sedimentary rocks	10 to 100
Groundwater, in areas of igneous rocks	30 to 150
Groundwater, in areas of sedimentary rocks	Larger than 1
Sea water	About 0.2

The relation between the depth of penetration and the electrode spacing is given in Figure 7.24, from which it can be seen that 50% of the total current passes above a depth equal to about half the electrode separation and 70% flows within a depth equal to the electrode separation. Analysis of the variation in the value of apparent resistivity with respect to electrode separation enables inferences about the subsurface formations to be drawn.

The resistivity method is based on the fact that any subsurface variation in conductivity alters the pattern of current flow in the ground and therefore changes the distribution of electric potential at the surface. Since the electrical resistivity of such factors as superficial deposits and bedrock differ from each other, the resistivity method may be used in their detection and to give their approximate thicknesses, relative positions and depths (Table 7.6). The first step in any resistivity survey should be to conduct a resistivity depth sounding at the site of a borehole in order to establish a correlation between resistivity and lithological layers. If a correlation cannot be established, then an alternative method is required.

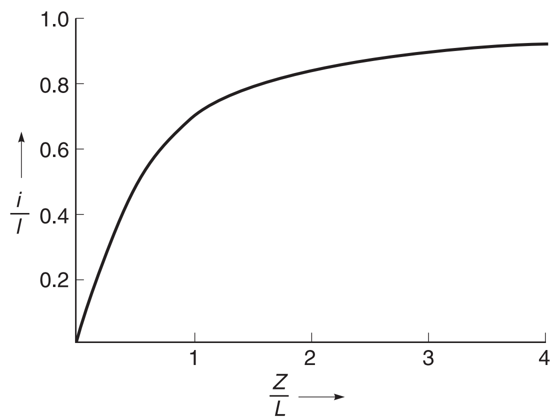


Figure 7.24

Fraction of total current, I, which passes above a horizontal plane at depth, Z, as a function of the distance, L, between two current electrodes.

Table 7.6. Resistivity values of some common rock types

Rock type	Resistivity (Ωm)
Topsoil	5–50
Peat and clay	8–50
Clay, sand and gravel mixtures	40–250
Saturated sand and gravel	40–100
Moist to dry sand and gravel	100–3000
Mudstones, marls and shales	8–100
Sandstones and limestones	100–1000
Crystalline rocks	200–10 000

The electrodes normally are arranged along a straight line, the potential electrodes being placed inside the current electrodes and all four are disposed symmetrically with respect to the centre of the configuration. The configurations of the symmetric type that are used most frequently are those introduced by Wenner and Schlumberger. Other configurations include the dipole–dipole and the pole–dipole arrays. In the Wenner configuration, the distances between all four electrodes are equal, whereas the distances between the potential electrodes and the centre of the array in the Schlumberger configuration are less than those between the current and potential electrodes (Fig. 7.25). The expressions used to compute the apparent resistivity, ρ_a, for the Wenner and Schlumberger configurations are as follows:

Wenner:

$$\rho_a = 2\pi aR \tag{7.9}$$

Schlumberger:

$$\rho_a = \frac{\pi(L^2 - l^2)}{2l} \times R \tag{7.10}$$

where a, L and l are explained in Figure 7.25 and R is the resistance reading.

Horizontal profiling is used to determine variations in apparent resistivity in a horizontal direction at a pre-selected depth. For this purpose, an electrode configuration, with fixed inter-electrode distances, is moved along a straight traverse, resistivity determinations being made at stations located at regular intervals. The length of the electrode configuration must be carefully chosen because it is the dominating factor in depth penetration. The data of a constant-separation survey consisting of a series of traverses arranged in a grid pattern, may

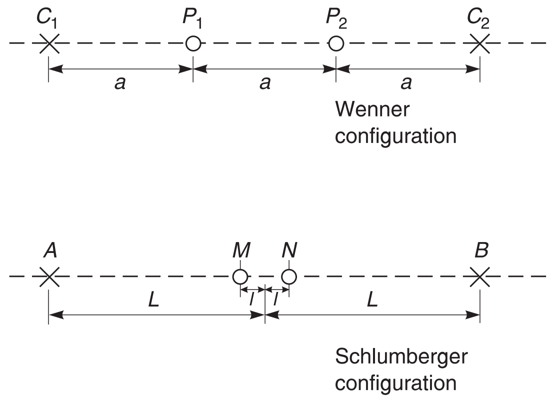


Figure 7.25

Wenner and Schlumberger configurations.

be used to construct a contour map of lines of equal resistivity. These maps often are useful in locating areas of anomalous resistivity such as gravel pockets in clay soils and the trend of buried channels. Even so, interpretation of resistivity maps as far as the delineation of lateral variations is concerned is mainly qualitative.

Electrical sounding furnishes information concerning the vertical succession of different conducting zones and their individual thicknesses and resistivities. For this reason, the method is particularly valuable for investigations on horizontally stratified ground. In electrical sounding, the mid-point of the electrode configuration is fixed at the observation station while the length of the configuration is increased gradually, that is, in the Wenner configuration, the distances between all four electrodes is increased progressively. Only the current electrodes are moved outwards about a station after each reading in the case of the Schlumberger configuration. As a result the current penetrates deeper and deeper, the apparent resistivity being measured each time the current electrodes are moved outwards. The readings, therefore, become increasingly affected by the resistivity conditions at advancing depths. The Schlumberger configuration is preferable to the Wenner configuration for depth sounding. The data obtained usually is plotted as a graph of apparent resistivity against electrode separation in the case of the Wenner array, or half the current electrode separation for the Schlumberger array. The electrode separation at which inflection points occur in the graph provide an idea of the depth of interfaces. The apparent resistivities of the different parts of the curve provide some idea of the relative resistivities of the layers concerned.

If the ground approximates to an ideal condition, then a quantitative solution, involving a curve-fitting exercise, should be possible. The technique requires a comparison of the observed curve with a series of master curves prepared for various theoretical models.

Generally, it is not possible to determine the depths to more than three or four layers. If a second layer is relatively thin and its resistivity much larger or smaller than that of the first layer, the interpretation of its lower contact will be inaccurate. For all depth determinations from resistivity soundings, it is assumed that there is no change in resistivity laterally. This is not the case in practice. Indeed, sometimes the lateral change is greater than that occurring with increasing depth, and so corrections have to be applied for the lateral effects when depth determinations are made.

Electromagnetic Methods

A wide variety of electromagnetic survey methods are available, each involving the measurement of one or more electric or magnetic field components induced in the ground by a primary field. A primary field is produced by a natural (transient) current source or an alternating current artificial source, and this field spreads out in space above and below the ground, inducing currents in subsurface conductors. Secondary electromagnetic fields are produced by these currents that distort the primary field. The resultant field differs from the primary field in intensity, phase and direction, and so can be detected by a suitable receiving coil. The secondary field induced in the subsurface conductor fades gradually when a transient primary field is switched off, fading being slower in media of higher conductivity. Hence, measurement of the rate at which the secondary currents fade and their field offers a means of detecting anomalously conducting bodies.

The terrain conductivity meter represents a means of measuring the conductivity of the ground. Electromagnetic energy is introduced into the ground by inductive coupling produced by passing an alternating current through a coil. The receiver also detects its signal by induction. The conductivity meter is carried along traverse lines across a site and can provide a direct continuous readout. Hence, surveys can be carried out quickly. Conductivity values are taken at positions set out on a grid pattern. Corrected values of conductivity can be plotted as contoured maps of conductivity. Where the thickness of overburden varies within fairly narrow limits and the conductivities of the overburden and bedrock do not change appreciably, the depth to bedrock can be estimated from standard curves. As these depth values are approximate, they need to be checked against borehole evidence or data obtained from more quantitative geophysical methods.

The very-low-frequency, VLF, method is the most widely used fixed-source method operating on a single frequency, making use of powerful radio transmitters. A disadvantage is that wave penetration is limited. The method also is affected by topography. The interpretation of VLF data generally is qualitative, and it frequently is used for reconnaissance work. The method is well suited to detecting near-vertical contacts and fracture zones.

As a consequence, the method has found particular application in site investigations for the delineation of faults.

The ground probing radar method is based upon the transmission of pulsed electromagnetic waves in the frequency range 1 to 1000 MHz. In this method, the travel times of the waves reflected from subsurface interfaces are recorded as they arrive at the surface, and the depth, Z, to an interface is derived from:

$$Z = VT/2 \tag{7.11}$$

where V is the velocity of the radar pulse and T is its travel time. The conductivity of the ground imposes the greatest limitation on the use of radar probing, that is, the depth to which radar energy can penetrate depends upon the effective conductivity of the strata being probed. This, in turn, is governed chiefly by the water content and its salinity. Furthermore, the value of effective conductivity is also a function of temperature and density, as well as the frequency of the electromagnetic waves being propagated. The least penetration occurs in saturated clayey materials or where the moisture content is saline. For example, attenuation of electromagnetic energy in wet clay and silt mean that depth of penetration frequently is less than 1 m. The technique appears to be reasonably successful in sandy soils and rocks in which the moisture content is non-saline. Rocks such as limestone and granite can be penetrated for distances of tens of metres and in dry conditions the penetration may reach 100 m. Ground probing radars have been used for a variety of purposes in geotechnical engineering, for example, the detection of fractures and faults in rock masses, the location of subsurface voids and the delineation of contaminated plumes.

Magnetic Methods

All rocks, mineral and ore deposits are magnetized to a lesser or greater extent by the Earth's magnetic field. As a consequence, in magnetic surveying, accurate measurements are made of the anomalies produced in the local geomagnetic field by this magnetization. The intensity of magnetization and hence the amount by which the Earth's magnetic field is changed locally depend on the magnetic susceptibility of the material concerned. In addition to the magnetism induced by the Earth's field, rocks possess a permanent magnetism that depends on their history.

Rocks have different magnetic susceptibilities related to their mineral content. Some minerals, for example, quartz and calcite are magnetized reversely to the field direction, and therefore have negative susceptibility and are described as diamagnetic. Paramagnetic minerals, which are the majority, are magnetized along the direction of magnetic field so that their susceptibility is positive. The susceptibility of the ferromagnetic minerals, such as magnetite, ilmenite, pyrrhotite and hematite, is a very complicated function of the field intensity. However, since the magnitudes of their susceptibility amount to 10 to 10^5 times the order of susceptibility of the paramagnetic and diamagnetic minerals, the ferromagnetic minerals can be found by magnetic field measurements.

If the magnetic field ceases to act on a rock, then the magnetization of paramagnetic and diamagnetic minerals disappears. However, in ferromagnetic minerals the induced magnetization is diminished only to a certain value. This residuum is called remanent magnetization and is of great importance in rocks. All igneous rocks have a very high remanent magnetization acquired as they cooled down in the Earth's magnetic field. In the geological past, during sedimentation in water, grains of magnetic materials were orientated by ancient geomagnetic fields so that some sedimentary rocks show stable remanent magnetization.

The strength of the magnetic field is measured in nanoteslas (nT), and the average strength of the Earth's magnetic field is about 50,000 nT. Obviously, the variations associated with magnetized rock formations are very much smaller than this. The intensity of magnetization and consequently the amount by which the Earth's magnetic field changes locally depends on magnetic susceptibility of the rocks concerned.

Aeromagnetic surveying has almost completely supplanted ground surveys for regional reconnaissance purposes. Accurate identification of the plan position of the aircraft for the whole duration of the magnetometer record is essential. The object is to produce an aeromagnetic map, the base map with transcribed magnetic values being contoured.

The aim of most ground surveys is to produce isomagnetic contour maps of anomalies to enable the form of the causative magnetized body to be estimated (Fig. 7.26). Profiles are surveyed across the trend of linear anomalies with stations, if necessary, at intervals of as little as 1 m. A base station is set up beyond the anomaly where the geomagnetic field is uniform. The reading at the base station is taken as zero, and all subsequent readings are expressed as plus-or-minus differences. Corrections need to be made for the temperature of the instrument as the magnets lose their effectiveness with increasing temperature. A planetary correction also is required that eliminates the normal variation of the Earth's magnetic field with latitude. Large metallic objects such as pylons are a serious handicap to magnetic investigation and must be kept at a sufficient distance as it is difficult to correct for them.

A magnetometer also may be used for mapping geological structures. For example, in some thick sedimentary sequences, it is sometimes possible to delineate the major structural features because the succession includes magnetic horizons. These may be ferruginous sandstones or shales, tuffs or basic lava flows. In such circumstances, anticlines produce positive and

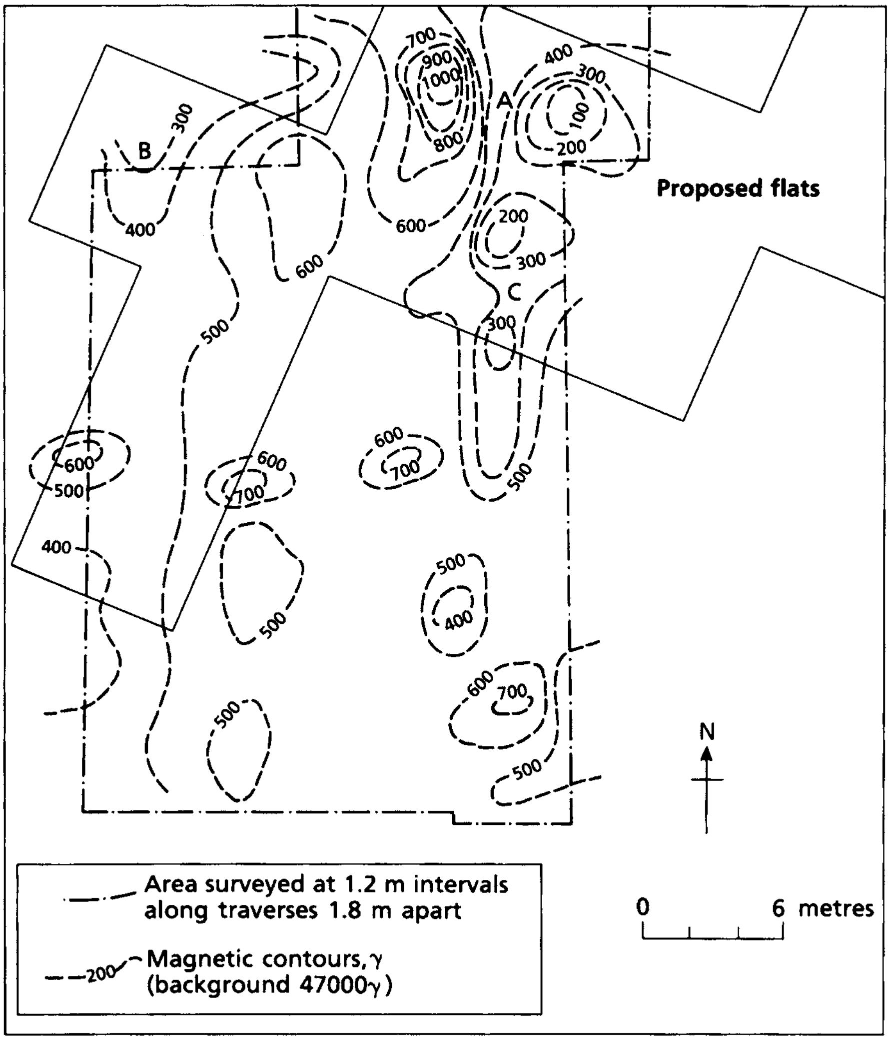


Figure 7.26

Magnetometer survey of a site for proposed flats in which mine shafts occurred at A, B and C (after Cripps et al., 1998).

synclines negative anomalies. Faults and dykes are indicated on isomagnetic maps by linear belts of somewhat sharp gradient or by sudden swings in the trend of the contours. However, in many areas the igneous and metamorphic basement rocks, which underlie the sedimentary sequence, are the predominant influence controlling the pattern of anomalies since they usually are far more magnetic than the sediments above. Where the basement rocks are brought near the surface in structural highs, the magnetic anomalies are large and characterised by strong relief. Conversely, deep sedimentary basins usually produce contours with low values and gentle gradients on isomagnetic maps.

Magnetic surveying has been used to detect abandoned mine shafts, a proton precession magnetometer normally being used (Bell, 1988c). A good subsurface magnetic contrast may be obtained if the shaft is lined with iron tubbing or with bricks or if the shaft is filled and the filling consists of burnt shale or ash or contains scrap iron. On the other hand, if a shaft is unfilled and unlined or lined with timber, then it may not give rise to a measurable anomaly.

Gravity Methods

The Earth's gravity field varies according to the density of the subsurface rocks, but at any particular locality, its magnitude also is influenced by latitude, elevation, neighbouring topographical features and the tidal deformation of the Earth's crust. The effects of these latter factors have to be eliminated in any gravity survey, where the object is to measure the variations in acceleration due to gravity precisely. This information then can be used to construct a contoured gravity map. In survey work, modern practice is to measure anomalies in gravity units (g.u. = 10^{-6} m s^{-2}). Modern gravity meters used in exploration measure not the absolute value of the acceleration due to gravity but the small differences in this value between one place and the next.

Gravity methods are used mainly in regional reconnaissance surveys to reveal anomalies that may be investigated subsequently by other methods. Since the gravitational effects of geological bodies are proportional to the contrast in density between them and their surroundings, gravity methods are particularly suitable for the location of structures in stratified formations. Gravity effects due to local structures in near surface strata may be partly obscured or distorted by regional gravity effects caused by large-scale basement structures. However, regional deep-seated gravity effects can be removed or minimized in order to produce a residual gravity map showing the effects of shallow structures that may be of interest.

A gravity survey is conducted from a local base station at which the value of the acceleration due to gravity is known with reference to a fundamental base where the acceleration due to gravity has been accurately measured. The way in which a gravity survey is carried out largely depends on the objective in view. Large-scale surveys covering hundreds of square kilometres, carried out in order to reveal major geological structures, are done by vehicle or helicopter with a density of only a few stations per square kilometre. For more detailed work such as the delineation of basic minor intrusions or the location of faults, spacing between stations may be as small as 20 m. Because gravity differences large enough to be of geological significance are produced by changes in elevation of several millimetres and of only 30 m in north-south distance, the location and elevation of stations must be established with very high precision.

Micro-gravity meters have been used to detect subsurface voids such as caverns in limestone, or abandoned mine shafts or shallow workings (Styles and Thomas, 2001). Gravity “lows” are recorded over voids, and they are more notable over air-filled than water- or sediment-filled voids.

Drillhole Logging Techniques

Drillhole logging techniques can be used to identify some of the physical properties of rocks. For example, the electrical resistivity method makes use of various electrode configurations down-the-hole. As the instrument is raised from the bottom to the top of the hole, it provides a continuous record of the variations in resistivity of the wall rock. In the normal or standard resistivity configuration, there are two potential electrodes and one current electrode in the sonde. The depth of penetration of the electric current from the drillhole is influenced by the electrode spacing. In a short normal resistivity survey, spacing is about 400 mm, whereas in a long normal survey, spacing generally is between 1.5 and 1.75 m. Unfortunately, in such a survey, because of the influence of thicker adjacent beds, thin resistive beds yield resistivity values that are much too low, while thin conductive beds produce values that are too high. The microlog technique may be used in such situations. In this technique the electrodes are very closely spaced (25–50 mm) and are in contact with the wall of the drillhole. This allows the detection of small lithological changes so that much finer detail is obtained than with the normal electric log (Fig. 7.30). A microlog is particularly useful in recording the position of permeable beds. If, for some reason, the current tends to flow between the electrodes on the sonde instead of into the rocks, then the laterolog or guard electrode is used. The laterolog 7 has seven electrodes in an array that focuses the current into the strata of the drillhole wall. The microlaterolog, a focused micro-device, is used in such a situation instead of the microlog.

A dipmeter generally is a four-arm side-wall micro-resistivity device. It measures small variations in the resistivity of a formation that allows the relative vertical shift of characteristic pattern variation produced by bedding planes, discontinuities or lithological changes to be used to determine, by aid of computer analysis, the attitude of a plane intersecting a drillhole. In this way, a fracture log can be produced.

Induction logging may be used when an electrical log cannot be obtained. In this technique, the sonde sends electrical energy into the strata horizontally and therefore only measures the resistivity immediately opposite the sonde, unlike in normal electrical logging where the current flows between electrodes. As a consequence, the resistivity is measured directly in

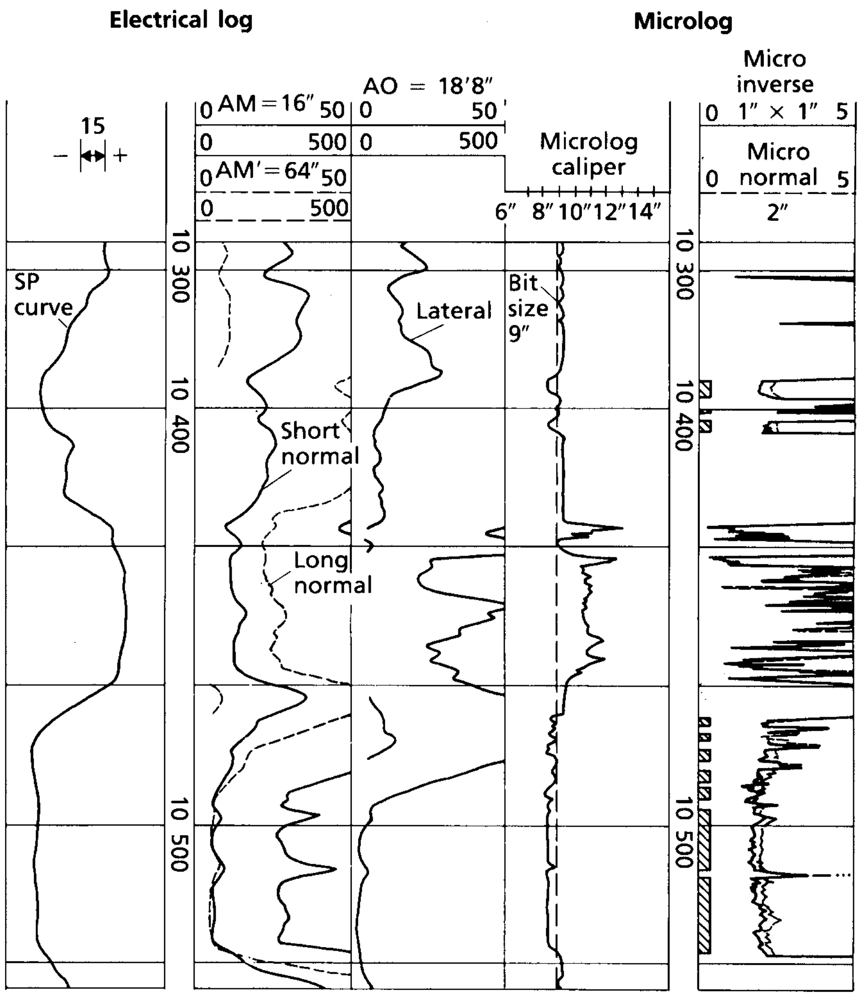


Figure 7.27

Drillhole logging curves. Microresistivity curves are shown on the right. Permeable portions of the penetrated section are indicated (cross-hatched bars) by extensions of the 50-mm micronormal curve beyond the microinverse. Note that the diameter of the bore, as recorded by the microlog calliper, is smaller than bit size where a mud filter cake is formed at the position of permeable beds. Standard electrical logs of the same stratigraphic interval are shown on the left for comparison.

an induction log, whereas in a normal electrical log, since the current flows across the stratal boundaries, it is measured indirectly from the electrical log curves. A gamma ray log usually is run with an induction log in order to reveal the boundaries of stratal units.

A spontaneous potential, SP, log is obtained by lowering a sonde down a drillhole that generates a small electric voltage at the boundaries of permeable rock units and especially between such strata and less permeable beds. For example, permeable sandstones show large SPs, whereas shales typically are represented by low values. If sandstone and shale are interbedded, then the SP curve has numerous troughs separated by sharp or rounded peaks, the widths of which vary in proportion to the thicknesses of the sandstones. Spontaneous potential logs frequently are recorded at the same time as resistivity logs. Interpretation of both sets of curves yields precise data on the depth, thickness and position in the sequence of the beds penetrated by the drillhole. The curves also enable a semi-quantitative assessment of lithological and hydrogeological characteristics to be made.

The sonic logging device consists of a transmitter–receiver system, transmitters and receivers being located at given positions on the sonde. The transmitters emit short, high-frequency pulses several times a second, and differences in travel times between receivers are recorded in order to obtain the velocities of the refracted waves. The velocity of sonic waves propagated in sedimentary rocks is largely a function of the character of the matrix. Normally, beds with high porosities have low velocities and dense rocks are typified by high velocities. Hence, the porosity of strata can be assessed. In the 3D sonic log, one transmitter and one receiver are used at a time. This allows both compressional and shear waves to be recorded, from which, if density values are available, the dynamic elastic moduli of the beds concerned can be determined. As velocity values vary independently of resistivity or radioactivity, the sonic log permits differentiation amongst strata that may be less evident on the other types of log.

Radioactive logs include gamma ray or natural gamma, gamma–gamma or formation density, and neutron logs. They have the advantage of being obtainable through the casing in a drillhole. On the other hand, the various electric and sonic logs can only be used in uncased holes. The natural-gamma log provides a record of the natural radioactivity or gamma radiation from elements such as potassium 40, and uranium and thorium isotopes, in the rocks. This radioactivity varies widely among sedimentary rocks, being generally high for clays and shales and lower for sandstones and limestones. Evaporites give very low readings. The gamma–gamma log uses a source of gamma rays that are sent into the wall of the drillhole. There they collide with electrons in the rocks and thereby lose energy. The returning gamma ray intensity is recorded, a high value indicating low electron density and hence low formation density. The neutron curve is a recording of the effects caused by bombardment of the strata with neutrons. As the neutrons are absorbed by atoms of hydrogen, which then emit gamma rays, the log provides an indication of the quantity of hydrogen in the strata around the sonde. The amount of hydrogen is related to the water (or hydrocarbon) content and therefore provides another method of estimating porosity. Since carbon is a good moderator of neutrons, carbonaceous rocks are liable to yield spurious indications as far as porosity is concerned.

The caliper log measures the diameter of a drillhole. Different sedimentary rocks show a greater or lesser ability to stand without collapsing from the walls of the drillhole. For instance, limestones may present a relatively smooth face slightly larger than the drilling bit whereas soft shale may cave to produce a much larger diameter. A caliper log is obtained along with other logs to help interpret the characteristics of the rocks in the drillhole.

Cross-Hole Methods

The cross-hole seismic method is based on the transmission of seismic energy between drillholes. In its simplest form, cross-hole seismic measurements are made between a seismic source in one drillhole (i.e. a small explosive charge, an air gun, a drillhole hammer, or an electrical sparker) and a receiver at the same depth in an adjacent drillhole. The receiver can either be a three-component geophone array clamped to the drillhole wall or a hydrophone in a liquid-filled drillhole to receive signals from an electric sparker in another drillhole similarly filled with liquid. The choice of source and receiver is a function of the distance between the drillholes, the required resolution and the properties of the rock mass. The best results are obtained with a high-frequency repetitive source. Generally, the source and receiver in the two drillholes are moved up and down together. Drillholes must be spaced closely enough to achieve the required resolution of detail and be within the range of the equipment. This is up to 400 m in clay, 160 m in chalk, and 80 m in sands and gravels. By contrast, because soft organic clay is highly attenuating, transmission is possible only over a few metres. These distances are for saturated material and the effective transmission is reduced considerably in dry superficial layers.

Such simple cross-hole seismic surveys are limited in the amount of data they produce. Hence, a system has been developed that uses a multitude of wave paths, thereby enabling the location, shape and velocity contrast relating to an anomaly or target in the rock mass between drillholes to be delineated in an unambiguous fashion. This is referred to as seismic tomography (tomography means a technique used to obtain an image of a selected plane section of a solid object). The method utilizes two or more drillholes, and possibly the ground surface, for the location of sources and detectors, the object being to derive one or more two-dimensional images of seismic properties within the rock mass (Jackson and McCann, 1997). Cross-hole seismic measurements provide a means by which the engineering properties of the rock mass between drillholes can be assessed. For example, the dynamic elastic properties can be obtained from the values of the compressional and shear wave velocities and the formation density (see the preceding text). Other applications include assessment of the continuity of lithological units between drillholes, identification of fault zones, assessment of the degree of fracturing and the detection of subsurface voids.

Electromagnetic and electrical resistivity techniques also have been used to produce tomographic imagery. For example, Corin et al. (1997) used drillhole radar tomography to assess the foundation conditions for a long viaduct to be constructed in limestone that was regarded as highly karstified. They concluded that cross-hole methods are probably the best tools available at present to provide the required detailed information, particularly in regions of karstic limestone, for good foundation design. Electrical resistance tomography is a relatively new geophysical imaging technique that uses a number of electrodes in drillholes, and sometimes at the ground surface, to image the resistivity of the subsurface.

Maps for Engineering Purposes

One of the important ways by which the geologist can be of service is by producing maps to aid the engineer, planner and others who are concerned with the development of land. A variety of maps can be produced from engineering geomorphological, environmental geological and engineering geological to geotechnical maps (Anon,1972). The distinction between these different types of maps is not always clear cut. Be that as it may, maps represent a means of storing and transmitting information, in particular, of conveying specific information about the spatial distribution of given factors or conditions. In addition, a map represents a simplified model of the facts, and the complexity of various geological factors can never be portrayed in its entirety. The amount of simplification required is governed principally by the purpose and scale of the map, the relative importance of particular geological factors or relationships, the accuracy of the data and on the techniques of representation employed.

The purpose of engineering geomorphological maps is to portray the surface form and the nature and properties of the materials of which the surface is composed, and to indicate the type and magnitude of the processes in operation. Surface form and pattern of geomorphological processes often influence the choice of a site. Hence, geomorphological maps give a rapid appreciation of the nature of the ground and thereby help the design of more detailed investigations, as well as focusing attention on problem areas. Such maps recognize landforms along with their delimitation in terms of size and shape (Fig. 7.28). Engineering geomorphological maps therefore should show how surface expression will influence an engineering project and should provide an indication of the general environmental relationship of the site concerned. If engineers are to obtain maximum advantage from a geomorphological survey, then derivative maps should be compiled from the geomorphological sheets. Such derivative maps generally are concerned with some aspect of ground conditions, such as landslip areas or areas prone to flooding or over which sand dunes migrate (Fig. 7.29).

The principal object during an engineering geomorphological survey is the classification of every component of the land surface in relation to its origin, present evolution and likely material properties. In other words, a survey should identify the general characteristics of the terrain of an area, thereby providing a basis for evaluation of alternative locations and avoidance of the

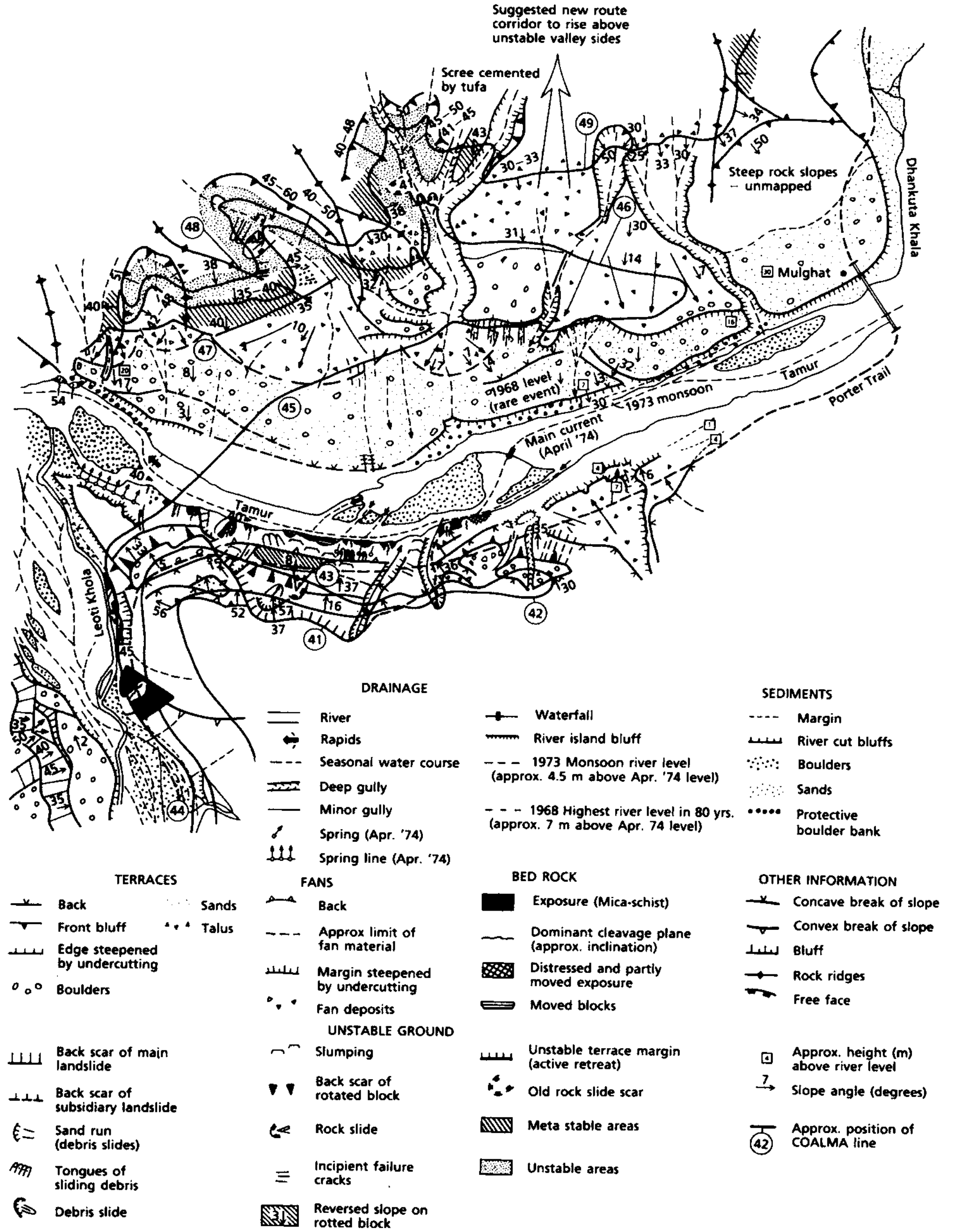


Figure 7.28

Engineering geomorphology map of the site and situation of a proposed bridge crossing on the Tamur River, eastern Nepal (after Brunsden et al., 1975). With kind permission of the Geological Society.

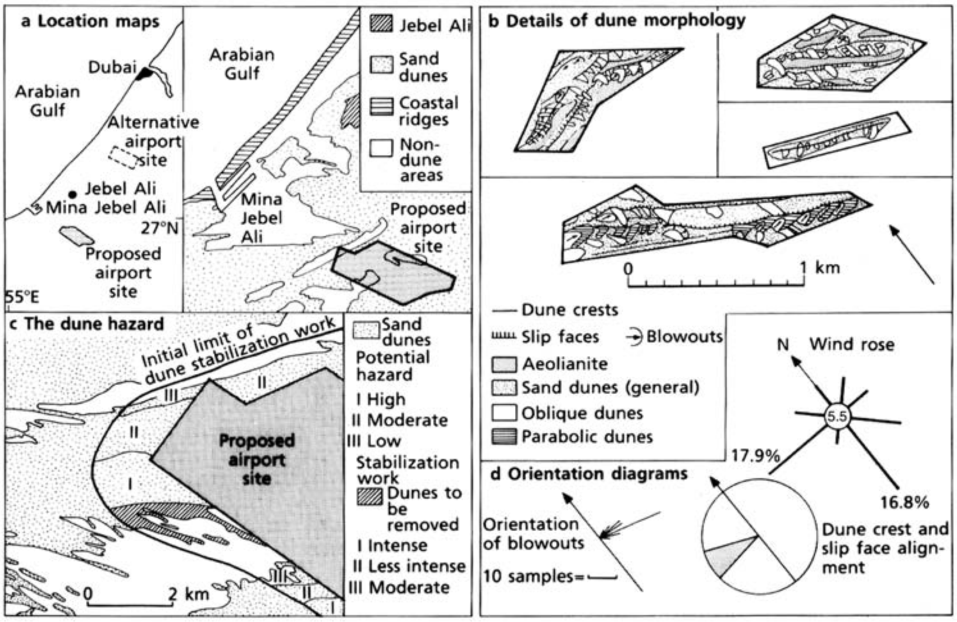


Figure 7.29

Geomorphological analysis of a proposed airport site in Dubai wih respect to the threat from mobile sand dunes (after Cooke et al., 1978). With kind permission of the Geological Society.

worst hazard areas. What is more, an understanding of the past and present development of an area is likely to aid prediction of its behaviour during and after construction operations. In addition, factors outside the site that may influence it, such as mass movement, should be identified and a synopsis of geomorphological development should be provided. Obtaining such information should facilitate the planning of a subsequent site investigation. For instance, it should aid the location of boreholes, and these hopefully will confirm what has been discovered by the geomorphological survey.

Environmental geology maps have been produced to meet the needs of planners (Forster et al., 2004). It is important that geological information should be understood readily by the planner and those involved in development. Unfortunately, conventional geological maps often are inadequate for the needs of such individuals (Section 7.1). Consequently, maps incorporating geological data are now being produced for planning and land-use purposes. Such maps are essentially simple and provide some indication of those areas where there are least geological constraints on development, and so they can be used by the planner or engineer at the feasibility stage of a project. The location of exploitable mineral resources also is of interest to planners. The obvious reason for presenting geological data in a way that can be understood by planners, administrators and engineers is that they then can seek appropriate professional advice and, in this way, bring about safer and more cost effective development and design of land, especially in relation to urban growth and redevelopment. In fact, two versions of environmental geology maps may be produced, one for the specialist and the other for the non-specialist.

Topics that are included on environmental geology maps vary, but may include solid geology, unconsolidated deposits, landslides, hydrogeology, mineral resources, contamination, shallow undermining and opencast workings, floodplain hazards, etc. (Fig. 7.30). Each aspect of geology can be presented as a separate theme on a basic or element map. Environmental potential maps are compiled from basic data maps. They present, in general terms, the constraints on development. They also can present those resources with respect to mineral, groundwater, or agricultural potential that might be used in development or that should not be sterilized by building over.

Engineering geological maps and plans provide engineers and planners with information that will assist them in the planning of land use and in the location and construction of engineering structures of all types. Such maps usually are produced on the scale of 1:10,000 or smaller, whereas engineering geological plans, being produced for a particular engineering

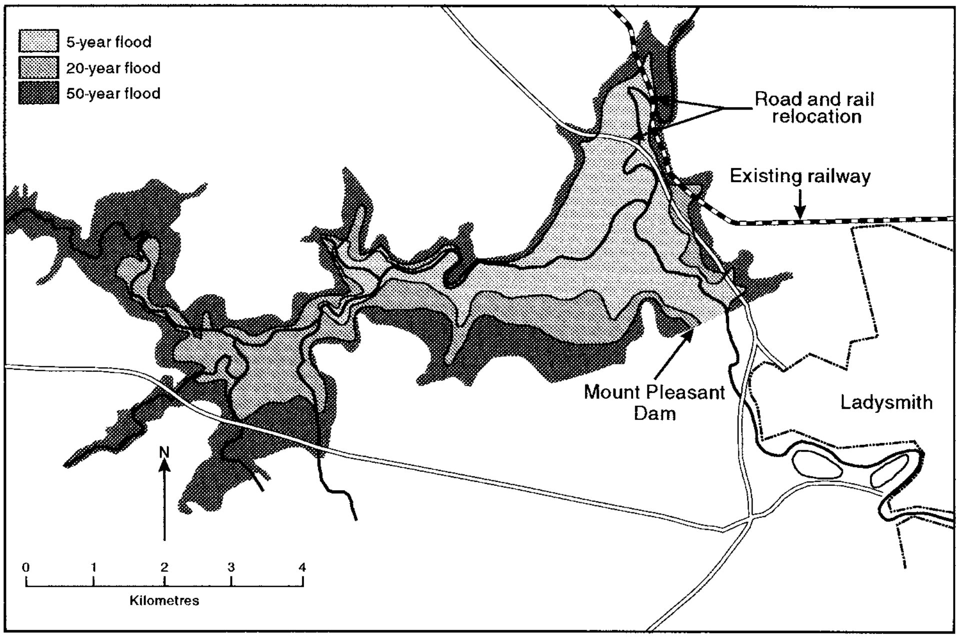


Figure 7.30

Proposed location of Mount Pleasant Dam to help protect Ladysmith from flooding, showing the 5-, 20- and 50- year flood boundaries, Natal, South Africa (after Bell and Mason, 1997).

purpose, have a larger scale. Engineering geological maps may serve a special purpose or a multipurpose (Anon, 1976). Special-purpose maps provide information on one specific aspect of engineering geology, for example, the engineering geological conditions at a dam site or along a routeway or for zoning for land use in urban development. Multipurpose maps cover various aspects of engineering geology.

Engineering geological maps should be accompanied by cross sections, and explanatory texts and legends. Detailed engineering geological information can be given, in tabular form, on the reverse side of the map (Fig. 7.31; Tables 7.7a and b). For example, a table of rock and soil characteristics summarizing the various rock and soil groups, listing their mode of occurrence, their thickness, their structure and their hydrogeological and geotechnical properties, may be provided. More than one map of an area may be required to record all the information that has been collected during a survey. In such instances a series of overlays or an atlas of maps can be produced. Preparation of a series of engineering geological maps can reduce the amount of effort involved in the preliminary stages of a site investigation, and may indeed allow site investigations to be designed for the most economical confirmation of the ground conditions.

Geotechnical maps and plans indicate the distribution of units, defined in terms of engineering properties. For instance, they can be produced in terms of index properties, rock quality or grade of weathering. A plan for a foundation could be made in terms of design parameters. The unit boundaries then are drawn for changes in the particular property. Frequently, the boundaries of such units coincide with stratigraphical boundaries. In other instances, as for example, where rocks are deeply weathered, they may bear no relation to geological boundaries. Unfortunately, one of the fundamental difficulties in preparing geotechnical maps arises from the fact that changes in physical properties of rocks and soils frequently are gradational. As a consequence, regular checking of visual observations by in situ testing or sampling is essential to produce a map based on engineering properties.

Geographical Information Systems

One means by which the power, potential and flexibility of mapping may be increased is by developing a geographical information system. Geographical information systems (GIS) represent a form of technology that is capable of capturing, storing, retrieving, editing, analyzing, comparing and displaying spatial information. For instance, Star and Estes (1990) indicated that a geographical information system consists of four fundamental components, namely, data acquisition and verification, data storage and manipulation, data transformation and analysis, and data output and presentation. The GIS software is designed to manipulate spatial data in order to produce maps, tabular reports or data files for interfacing with numerical models.

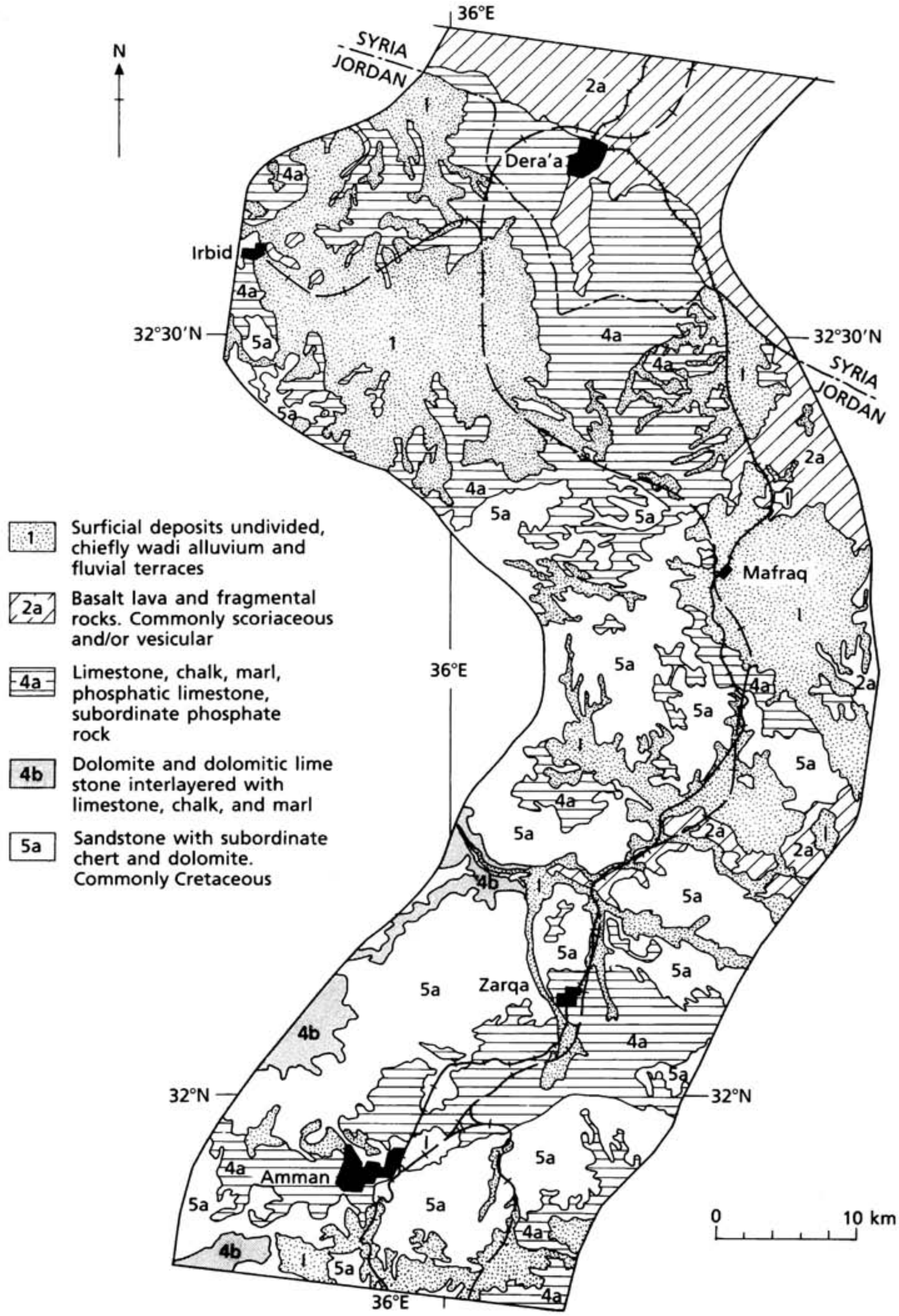


Figure 7.31

Segment for the engineering geological map for the Hijaz Railway in Jordan (after Briggs, 1987). Engineering geology and characteristics shown in Table 7.7a and b. With kind permission of the Association of Engineering Geologists.

Table 7.7a. Excerpts from the engineering geology table illustrating the variety of materials in the study area for the Hijaz Railway. Symbols 1, 2a, 4a, 4b and 5a are shown in Figure 7.31 (after Briggs, 1987)

Map symbol	Geological description	Distribution	Map segments	Engineering characteristics	Suitability as source of material for:	Moderate water supply favourability in shallow aquifers	Topographic expression
1	Surficial deposits undivided, chiefly wadi alluvium and fluvial and marine terraces	Most common in Saudi Arabia and southern Jordan	Present on most map segments	Excavation: easy Stability: poor Strength: fair Tunnel support: maximum	Ballast – 0 Coarse aggregate – + Sand – +++ Embankments – 0 Riprap –0	Fair to good with seasonal fluctuations. Coastal areas poor	Generally flat, locally steeply dissected
2a	Basalt lava and fragmental rocks. Commonly scoriaceous and/or vesicular	Widespread in southern Syria. Locally elsewhere	01, 02, 13, 19, 20 and 28	Excavation: difficult Stability: good Strengh: good Tunnel support: moderate	Ballast – + Coarse aggregate – + Sand – + Embankments – ++ Riprap – +	Generally poor. Locally fair to good, depending on interlayering	Flat to mountainous. Surfaces commonly bouldery
3c	Sandstone and conglomerate with limestone and marl. Loosely cemented.Locally hard	Along coastal plain between A1 Wajh and Yanbu, Saudi Arabia	26, 27, 28 and 30	Excavation: intermediate Stability: fair Strength: fair Tunnel support: moderate to maximum	Ballast – 0 Coarse aggregate – 0 Sand – + Embankments – ++ Riprap – 0	Poor	Flat to rolling, locally hilly and dissected
4a	Limestone, chalk, marl, phosphatic limestone, subordinate phosphate rock	Widespread in Jordan	02–05 and 29	Excavation: difficult Stability: fair to good Strength: fair to good	Ballast – 0 Coarse aggregate – + Sand – 0 Embankments – + Riprap – +	Generally poor	Hilly, locally rolling or mountainous

Continued

Table 7.7a. Excerpts from the engineering geology table illustrating the variety of materials in the study area for the Hijaz Railway. Symbols 1, 2a, 4a, 4b and 5a are shown in Figure 7.31 (after Briggs, 1987)—cont'd

Map symbol	Geological description	Distribution	Map segments	Engineering characteristics	Suitability as source of material for:	Moderate water-supply favourability in shallow aquifers	Topographic expression
				Tunnel support: moderate to minimum			
4b	Dolomite and dolomitic limestone interlayered with limestone, chalk, and marl	Central Jordan	02 and 03	Excavation: moderately difficult Stability: fair to good Strength: fair to good Tunnel support: moderate to minimum	Ballast – + Coarse aggregate – + Sand – 0 Embankments – ++ Riprap – +	Generally poor	Hilly, locally mountainous
5a	Sandstone with subordinate chert and dolomite. Commonly calcareous	Widespread in Jordan	02–05 and 29	Excavation: moderately difficult Stability: fair to good Strength: good Tunnel support: moderate to minimum	Ballast – 0 Coarse aggregate – 0 Sand – + Embankments – ++ Riprap – 0	Poor to fair	Hilly to mountainous

6b	Chiefly andesite lava and fragmental rocks. Common medium-grade metamorphism, greenstone	Widespread in Hijaz Mountains	10–13, 18–25, 27, 30 and 31	Excavation: difficult Stability: good Strength: good Tunnel support: minimum	Ballast – +++ Coarse aggregate – +++ Sand – 0 Embankments – ++ Riprap – +++	Poor	Core of Hijiz Mountains. Relief locally greater than 2000m
7b	Early and altered granites, granodiorite, quartz monzonite. Includes some gneiss	Common in the Hijaz Mountains and southern Jordan	10–13 and 19–31	Excavation: difficult Stability: good Strength: good Tunnel support: minimum to moderate	Ballast – + Coarse aggregate – + Sand – + Embankments – ++ Riprap – ++	Poor	Chiefly mountainous. Mostly more resistant than other intrusive rocks

Table 7.7b. Key to the engineering characteristics column (after Briggs, 1987)

Excavation facility	Stability of cut slopes	Foundation strength	Tunnel support requirements
Easy – can be excavated by hand tools or light power equipment. Some large boulders may require drilling and blasting for their removal. Dewatering and bracing of deep excavation walls may be required	Good – these rocks have been observed to stand on essentially vertical cuts where jointing and fracturing are at a minimum. However, moderately close jointing or fracturing is common, so slopes not steeper than 4:1 (vertical: horizontal) are recommended. In deep cuts, debris-catching benches are recommended	Good – bearing capacity is sufficient for the heaviest classes of construction, except where located on intensely fractured or jointed zones striking parallel to and near moderate to steep slopes	Minimum – support probably required for less than 10% of length of bore, except where extensively fractured
Moderately easy – probably rippable by heavy power equipment at least to weathered rock – fresh rock interface and locally to greater depth	Fair – cut slopes ranging from 2:1 to 1:1 are recommended; flatter where rocks are intensely jointed or fractured. Rockfall may be frequent if steeper cuts are made. Locally, lenses of harder rock may permit steeper cuts	Fair – choice of foundation styles is largely dependent on packing of fragments, clay content, and relation to the water table. If content of saturated clay is high, appreciable lateral movement of clay may be expected under heavy loads. If packing is poor, settling may occur	Moderate – support may be required for as much as 50% of length of bore, more where extensively fractured
Intermediate – probably rippable by heavy power equipment to depths chiefly limited by the manoeuverability of the equipment. Hard rock layers or zones of hard rock may require drilling and blasting	Poor – flatter slopes are recommended. Some deposits commonly exhibit a deceptive temporary stability, sometimes standing on vertical or near-vertical cuts for periods ranging from hours to more than a year	Poor – foundations set in underlying bedrock are recommended for heavy construction, with precautions taken to guard against failure due to lateral stress	Maximum – support probably required for entire length of bore
Moderately difficult – probably require drilling and blasting for most deep excavations, but locally may be ripped to depths of several metres			
Difficult – probably require drilling and blasting in most excavations except where extensively fractured or altered			

Table 7.8. Advantages and disadvantages of GIS

Advantages	**Disadvantages**
1. A much larger variety of analysis and techniques are available. Because of the speed of calculation, complex techniques requiring a large number of map overlays and table calculations become feasible.	1. A large amount of time is needed for data entry. Digitizing is especially time consuming.
2. It is possible to improve models by evaluating their results and adjusting the input variables. Users can achieve the optimum results by a process of trial and error, running the models several times, whereas it is difficult to use these models even once in the conventional manner. Therefore, more accurate results can be expected.	2. There is a danger in placing too much emphasis on data analysis as such at the expense of data collection and manipulation based on professional experience. A large number of different techniques of analysis are theoretically possible, but often the necessary data are missing. In other words, the tools are available but cannot be used because of the lack of certainty of input data.
3. In the course of a hazard assessment project, the input maps derived from field observations can be updated rapidly when new data are collected. Also, after completion of the project, the data can be used by others in an effective manner.	

An important feature of a GIS is the ability to generate new information by the integration of existing diverse data sets sharing a compatible referencing system. Data can be obtained from remote sensing imagery, aerial photographs, aeromagnetometry, gravimetry, and various types of maps. This data is recorded in a systematic manner in a computer database. Each type of data input refers to the characteristics of recognizable point, linear or spacial geographical features. Details of the features usually are stored in either vector (points, lines and polygons) or raster (grid cell) formats. The manipulation and analysis of data allows it to be combined in various ways to evaluate what will happen in certain situations.

Currently, there are many different geographical information systems available, ranging from public domain software for PCs to very expensive systems for mainframe computers. Since most data sets required in environmental or engineering geology data processing are still relatively small, they can be accommodated readily by inexpensive PC based GIS applications. The advantages of using GIS compared with conventional spatial analysis techniques have been reviewed by Burrough (1986) and are summarised in Table 7.8.

An ideal GIS for many engineering geological situations combines conventional GIS procedures with image processing capabilities and a relational database. Because frequent map overlaying, modelling and integration with scanned aerial photographs and satellite images are required, a raster system is preferred. The system should be able to perform spatial analysis on multiple-input maps and connected attribute data tables. Necessary GIS functions include map overlay, reclassification and a variety of other spatial functions incorporating logical, arithmetic, conditional and neighbourhood operations. In many cases, modelling requires the iterative application of similar analyses using different parameters. Consequently, the GIS should allow for the use of batch files and macros to assist in performing these iterations.

Mejía-Navarro and Gracia (1996) referred to several attempts to use GIS for geological hazard and vulnerability assessment. These were especially in relation to the assessment of landslide, seismic and fluvial hazards. They then went on to describe a decision support system for planning purposes that evaluates a number of variables by use of GIS. This integrated computer support system, termed Integrated Planning Decision Support System (IPDSS) was designed to assist urban planning by organizing, analyzing, and evaluating existing or needed spatial data for land-use planning. The system incorporates GIS software that allows comprehensive modelling capabilities for geological hazards, vulnerability and risk assessment. The IPDSS uses data on topography, aspect, solid and superficial geology, structural geology, geomorphology, soil, land cover and use, hydrology and floods, and historical data on hazards. As a consequence, it has been able to delineate areas of high risk from those where future urban development could take place safely and is capable of producing hazards susceptibility maps.

Geology, Planning and Development

Introduction

The ultimate objective of planning is to determine a particular course of action. Although the policy that develops from planning embodies a particular course of action, planning proposals are often controversial in that they may offend one or more sections of the community. Hence, in the last analysis, planning policies are the prerogatives of government since legislation is necessary to put them into effect.

Land-use planning represents an attempt to resolve conflicts between the need to utilize land and at the same time to protect the environment. Hence, planners have to assess the advantages, disadvantages, costs and benefits of development. In the first instance, land-use planning involves the collection and evaluation of relevant data from which plans can be formulated. The policies that result also have to take account of economic, sociological and political influences. As far as geology is concerned, sufficient data should be provided to planners and engineers so that, ideally, they can develop the environment in harmony with nature. Geological information is required at all levels of planning and development from the initial identification of a social need to the construction stage. Even after construction, further involvement may be necessary in the form of advice on hazard monitoring, maintenance or remedial works.

Over recent years, public concern regarding the alteration and degradation of the environment has caused governmental and planning authorities to become more aware of the adverse effects of indiscriminate development. As a result, laws have been passed to help protect the environment from spoilation. Most policies that deal with land use are concerned with either those processes that represent threats to life, health or property, including, for instance, hazardous events; pollution of air or water; the exploitation, protection or conservation of natural resources; or the restoration of despoiled areas.

Laws are now in force in many countries that require the investigation and evaluation of the consequences to the environment of the development of large projects. Hence, an environmental assessment can be defined as an activity designed to identify and predict the impact on society of legislative proposals, policies, programmes, projects and operational procedures, as well as to interpret and communicate information about such impacts. As such, an environmental assessment is a multidisciplinary process with a social component, as well as a technical component. Environmental assessment now forms an established part of good development planning and was formulated initially in the United States by way of the National Environmental Policy Act (1969), which entails the preparation of a formal statement on the consequences of any major federal activity on the environment. In other words, an environmental impact assessment is required, and this is accompanied by other relevant documents that are necessary to the decision making process. An environmental impact assessment therefore usually involves the description of a proposed scheme, its impact on the environment, particularly noting any adverse effects and alternatives to the proposed scheme. The aim of the environmental impact process is to improve the effectiveness of existing planning policy. It should bring together the requirements of development and the constraints of the environment, so that conflicts are minimized. Thus, the primary purpose of environmental assessment is to aid decision making by national or local government by providing objective information relating to the effects on the environment of a project or course of action. A number of associated objectives follow from this. First, an environmental impact assessment must provide objective information on which decisions can be made. Therefore, it involves data collection relevant to environmental impact prediction. The information it contains must be comprehensive so that, after analysis and interpretation, it not only informs but also helps direct development planning. Environmental assessment must be relevant and must involve comparative analyses of reasonable alternatives. Plans must be analyzed, so as to ensure that the benefits of a project are fully appreciated. The negative effects, as well as the positive ones, of development proposals have to be conveyed to the decision makers. Solutions should be proposed to any problems that may arise as a result of interactions between the environment and the project. Geologists are invariably involved in environmental impact assessments.

Since land use inevitably involves different development of particular areas, some type of land classification constitutes the basis on which land-use planning is carried out. However, land should also be graded according to its potential uses and capabilities. In other words, indices are required to assess the environmental status of natural resources and their potential.

Legget (1987) stressed the importance of geology in planning physical facilities and individual structures, and in the wise use of land. He pointedly noted the obvious, that land is the surface expression of underlying geology, and that land use planning can only be done with satisfaction if there is a proper understanding of the geology concerned. Legget further stated that the development of land must be planned with the full realization of the natural forces that have brought it to its present state, taking into account the dynamic character of nature so that development does not upset the delicate balance any more than is essential. Geology therefore must be the starting point of all planning.

It therefore is important that geological information should be understood by the planner, developer and engineer. Hence, various types of maps incorporating geological data are being produced for planning and development purposes, and may have accompanying explanatory memoirs (Fig. 8.1; Culshaw et al., 1990). Such maps should be readily understandable and provide some indication of those areas where there are least geological constraints on development, and hence allow their use by the planner or engineer at the feasibility stage of a project (see Chapter 7).

Reports containing comprehensive suites of maps are now produced that can be used by local and central planners, and also represent useful sources of information for civil engineers,

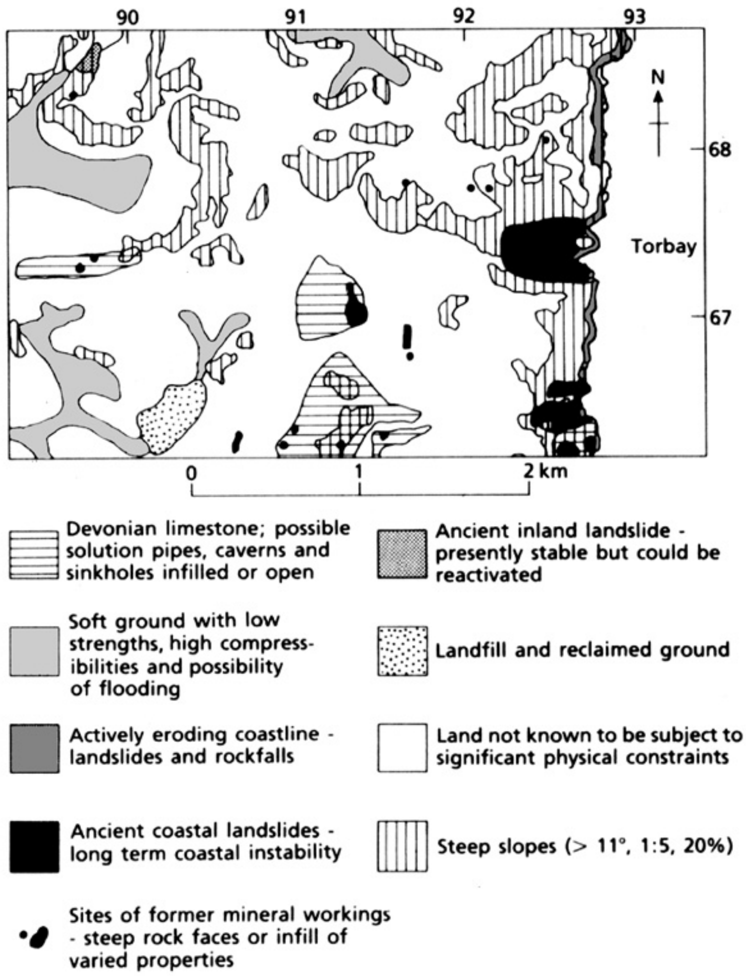


Figure 8.1

Ground characteristics for planning and development for part of the Torbay area of south west England (after Culshaw et al., 1990).

developers, and mineral extraction companies. Such maps should help planners avoid making poor decisions at an early stage and allow areas to be developed sequentially (e.g. mineral resources can be developed before an area is built over). In addition, they can facilitate the development of a multi-use plan of a particular area and avoid one factor adversely affecting another. For example, in Britain this began in the early 1980s, following a pilot study of the Glenrothes area of Fife, Scotland. This study produced 27 separate maps covering such aspects as stratigraphy and lithology of bedrock, superficial deposits, rockhead contours, engineering properties, mineral resources and workings, groundwater conditions and landslip potential.

Geological Hazards, Risk Assessment and Planning

Obviously, one of the aspects of planning that intimately involves geology is the control or reduction of the effects of geological processes or geohazards that work against the interests of humans. Geological hazards can be responsible for devastating large areas of the land surface and so can pose serious constraints on development. However, geological processes such as volcanic eruptions, earthquakes, landslides and floods cause disasters only when they impinge upon people or their activities. Even so, as the global population increases, the significance of geohazards is likely to rise. In terms of financial implications, it has been estimated that natural hazards cost the global economy over $50,000 million per year. Two thirds of this sum is accounted for by damage, and the remainder represents the cost of predicting, preventing and mitigating against disasters.

Geological hazards vary in their nature and can be complex. One type of hazard, for example, an earthquake, can be responsible for the generation of others such as liquefaction of sandy or silty soils, landslides or tsunamis. Certain hazards such as earthquakes and landslides are rapid onset hazards and so give rise to sudden impacts. Others such as soil erosion and subsidence due to the abstraction of groundwater may take place gradually over an appreciable period of time. Furthermore, the effects of natural geohazards may be difficult to separate from those attributable to human influence. In fact, modification of nature by humans often increases the frequency and severity of natural geohazards, and, at the same time, these increase the threats to human occupancy.

The development of planning policies for dealing with geohazards requires an assessment of the severity, extent and frequency of the geohazard in order to evaluate the degree of risk. The elements at risk are life, property, possessions and the environment. Risk involves quantification of the probability that a hazard will be harmful, and the tolerable degree of risk depends on what is being risked, life being much more important than property. The risk to society can be regarded as the magnitude of a hazard multiplied by the probability of its occurrence.

If there are no mitigation measures, no warning systems and no evacuation plans for an area that is subjected to a recurring geohazard, then such an area has the highest vulnerability.

The effects of a disaster may be lessened by reduction of vulnerability. Vulnerability analyses comprising risk identification and evaluation should be carried out in order to make rational decisions on how best the effects of potentially dangerous events can be reduced or overcome. Short-term forecasts, a few days ahead of an event, may be possible and complement relief and rehabilitation planning. In addition, it may be possible to reduce the risk of disaster by a combination of preventative and mitigative measures. To do this successfully, the patterns of behaviour of the geological phenomena posing the hazards need to be understood and the areas at risk identified. Then, the level of potential risk may be decreased and the consequences of disastrous events mitigated against. Mitigation measures may be structural or technological on the one hand or regulatory on the other. In other words, the risks associated with geological hazards may be reduced by control measures carried out against the hazard-producing agent; monitoring and warning systems that allow evacuation; restrictions on development of land; and the use of appropriate building codes together with structural reinforcement of property.

The net impact of a geohazard can be looked upon in terms of the benefits derived from inhabiting an area in which the hazard occurs, minus the costs involved in mitigation measures. The risk to society can be quantified in terms of the number of deaths attributable to a particular geohazard in a given period of time and the resultant damage to property. The costs involved can then be compared to the costs of hazard mitigation. Unfortunately, however, not all risks and benefits are readily amenable to measurement and assessment in financial terms. Even so, risk assessment is an important objective in decision making by planners because it involves the vulnerability of people and property on the basis of probability of occurrence of an event. The aims of risk analysis are to improve the planning process, as well as to reduce vulnerability and mitigate against damage. An objective assessment of risk is obtained from a statistical assessment of instrumentally obtained data and/or data gathered from past events. In a conventional statistical assessment, probability expresses the relative frequency of occurrence of an event, based on data collected from various sources. A number of computer programs are available to carry out such analyses.

Hazard Maps

Any spacial aspect of hazard can be mapped, provided there is sufficient information on its distribution. Hence, when hazard and risk assessments are made over a large area, the results can be expressed in the form of hazard and risk maps. An ideal hazard map should provide information relating to the spacial and temporal probabilities of the hazard mapped. In addition to data gathered from surveys, hazard maps are frequently compiled from historical data related to past hazards that have occurred in the area concerned. The concept is based on the view that past hazards provide some guide to the nature of future hazards. Hazard zoning maps usually provide some indication of the degree of risk involved with a particular geological hazard. The hazard is often expressed in qualitative terms such as high, medium and low. These terms, however, must be adequately described so that their meaning is understood. The variation in intensity of a hazard from one location to another can be depicted by risk mapping. The latter attempts to quantify the hazard in terms of potential victims or damage. Therefore, risk mapping attempts to estimate the location, probability and relative severity of probable hazardous events so that potential losses can be estimated, mitigated against or avoided. Specific risk zoning maps divide a region into zones indicating exposure to a specific hazard. For example, a method of mapping areas prone to geological hazards has been developed that uses map units based primarily on the nature of the potential hazards associated with them. The resultant maps, together with their explanation, can be combined with a land-use/geological hazard area matrix to provide some idea of the problems that may arise in the area represented by the individual map. For example, the landslide hazard map of the Crested Butte-Gunniston area, Colorado, developed by Soule (1980), provides an illustration of this method (Fig. 8.2). The map, together with its explanation, are combined with a land-use/geological hazard area matrix that provides some idea of the engineering problems that may arise in the area. For instance, the matrix indicates the effects of any changes in slope or the mechanical properties of rocks or soils, and attempts to evaluate the severity of hazard for various land uses. The map attempts to show the factors within individual map units that have the most significance as far as potential hazard is concerned. The accompanying matrix outlines the problems likely to be encountered as a result of human activity.

Microzonation has been used to depict the spacial variation of risk in relation to particular areas. Ideally, land-use should adjust to the recommendations suggested by any such microzoning so that the impacts of the geohazard events depicted have a reduced or minimum influence on people, buildings and infrastructure. Single geohazard–single purpose, single geohazard–multiple purpose and multiple hazard–multiple purpose maps can be distinguished. The first type of map obviously is appropriate where one type of geohazard occurs, and the effects are relatively straightforward. Single geohazard–multiple purpose maps are prepared where the geohazard is likely to affect more than one activity and shows the varying intensity of and the response to the hazard. The most useful map as far as planners are concerned is the multiple geohazard–multiple purpose map. Such a map can be used for risk analysis and assessment of the spatial variation in potential loss and damage. Both the geohazard and the consequences should be quantified, and the relative significance of each hazard compared. The type and amount of data involved lends itself to processing by a geographical information system.

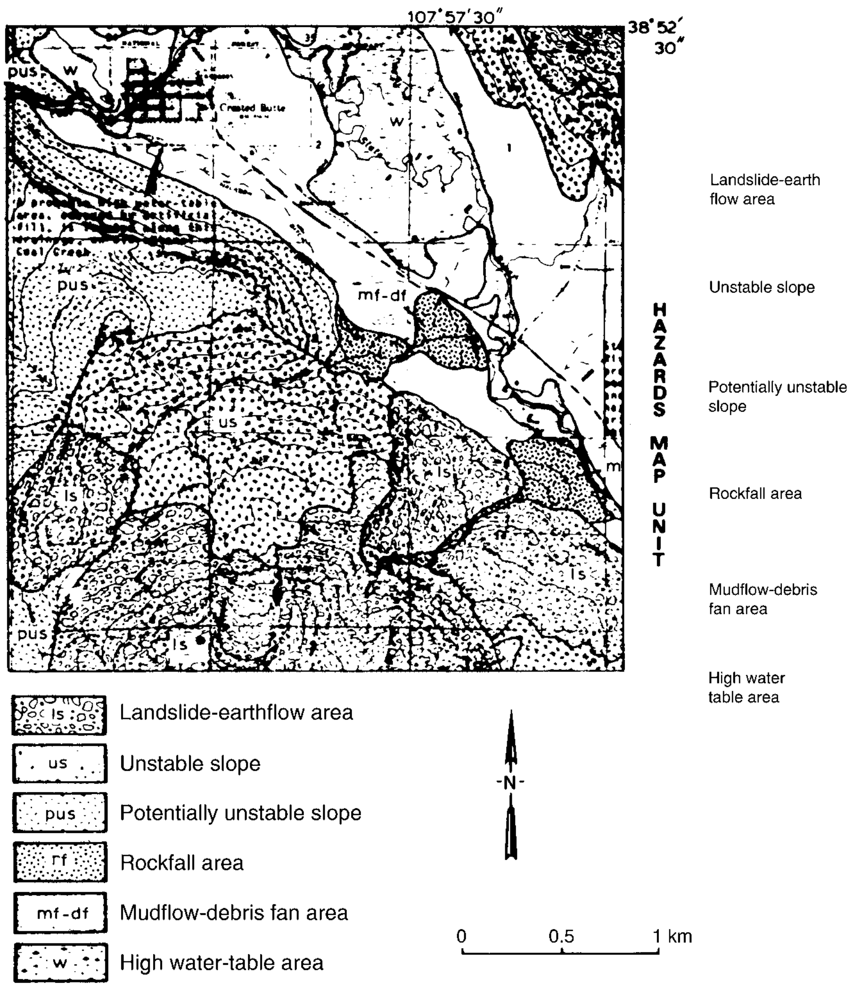


Figure 8.2

(a) An example of engineering geological hazard mapping in the Crested Butte-Gunnison area (after Soule, 1980). (b) The matrix is formatted so as to indicate to the map user that several geological and geology-related factors should be considered when contemplating the land use in a given type of mapped area. This matrix also can serve to recommend additional types of engineering geological studies that may be needed for a site. Hence, the map can be used to model or anticipate the kinds of problems that a land-use planner or land developer may have to overcome before a particular activity is permitted or undertaken.

Natural Geological Hazards and Planning

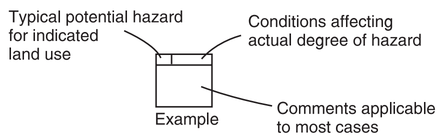
Volcanic Activity

Volcanic eruptions and other manifestations of volcanic activity are variable in type, magnitude, duration and significance as a geohazard. Most volcanism is associated with the boundaries of the crustal plates of the Earth, and the activity produced by a volcano varies according to the

Residential Development							
High density	Low density	Roads	Utilities	Open space recreational complexes including ski areas, but not associated structures	Commercial and industrial development, including larger residential buildings such as condominiums and apt buildings	Low-value lightweight agricultural buildings	Agriculture uses grazing and similar
3 + ABCDEFH	3 ABCDFH	3 + ABCDEFH	2 ABCEFH	1 + AB	3 + ABCFH	2 + AFH	1 + ABE
Remedial engineering typically is prohibitively expensive	May be possible with careful engineering.	Typically not feasible without careful engineering.	Compatible with open-space land use.	Commonly feasible. Maintenance costs may be high.	Maintenance costs probably will be high.	Maintenance costs may be high.	Usually minor problems except for irrigation ditches and fences.
3 ACDEFH	2 + ACDEFH	3 ACDEFH	1 + ACFH	1 AF	3 ACDEFH	2 AFH	1 ADE
Remedial engineering usually necessary.	Remedial engineering may be necessary.	Remedial engineering usually necessary.	Remedial engineering may be necessary.	Commonly feasible.	Remedial engineering necessary.	Remedial engineering necessary.	Usually minor problems except where ditch leakage causes earthflows.
3 CDEF	2 CDEF	2 + ABCDEFH	1 CDE	0 F	2 ACFH	2 ACFH	0 E
Remedial engineering may be necessary.	Remedial engineering may be necessary.	May experience difficulties without careful planning/ engineering.	Careful planning can minimise hazard.	Typically no difficulties.	Remedial engineering necessary.	Remedial engineering necessary.	Usually minor problems except in areas of intense cultivation of hillslopes.
3 + ABCFH	3 ABCFH	3 ABCFH	2 ABCH	1 + B	2 ABCH	2 + ABCFH	1 B
Rarely compatible without elaborate and expensive mitigation.	Careful siting typically necessary to minimise hazard.	Remedial engineering can minimise hazard.	Careful planning can minimise hazard.	Careful siting can minimise hazard.	Maintenance costs probably will be high.	Careful siting can minimise hazard.	Usually few or minor problems.
3 + BCDFH	3 BCDFH	3 BDH	2 BFH	2 B	2 BCDFH	2 BDFH	0 B
Rarely compatible without elaborate and expensive mitigation.	Rarely compatible without elaborate and expensive mitigation.	Compatible only with elaborate and expensive mitigation.	Possibly excessive maintenance necessary.	Commonly feasible if rise is acceptable.	Occasional very high maintenance costs can be expected.	Occasional very high maintenance costs can be expected.	Usually few or minor problems.
3 GH	2 + GH	3 GH	0 G	1 G	2 + GH	2 GH	0 G
Basements and septic tank sewage disposal usually not feasible.	Basements and septic tank sewage disposal usually not feasible.	Usually difficult depends on type of development. Flood plain determination may be necessary.	Some remedial engineering may be necessary in unusual cases.	Usually little difficulty. Possibility of flood damage.	May require special construction techniques remedial engineering.	May require special construction techniques remedial engineering.	Desirable for many kinds of agriculture.

EXPLANATION OF CHART SYMBOLS

3 High hazard
2 Moderate hazard
1 Low hazard
0 Very low, if any hazard



MEANING OF LETTER SYMBOLS

A Especially severe on slopes greater than 30°
B Slope movement intermittent dependent on variation in weather or other factors.
C Oversteepening or cutting of slopes can increase hazard greatly.
D Artificial or natural increase in ground moisture can increase hazard greatly.
E Removal of natural vegetation can increase hazard greatly.
F Hazard may decrease considerably as slope decreases.
G Varies seasonably.
H Detailed engineering geology studies necessary during preplanning stages of development.

Figure 8.2, cont'd

type of plate boundary. Generally, the activity associated with diverging plate boundaries entails a relatively gentle upwelling of lava. Since the lava has low viscosity, the eruptions tend to be non-violent in nature, although large quantities of ash and lava may be thrown into the air. Furthermore, the lava may flow large distances before it cools. In contrast to this, the activity associated with converging plate boundaries tends to be explosive in nature. Owing to the high viscosity of the lava, volcanic vents can become blocked so that subterranian pressures build up. The sudden release of these pressures during an eruption can give rise to nueés ardentes, the ejection of large quantities of material to great heights and localized seismicity.

In any assessment of risk due to volcanic activity, the number of lives at stake, the capital value of the property and the productive capacity of the area concerned have to be taken into account. Evacuation from danger areas is possible if enough time is available. However, the vulnerability of property is frequently close to 100% in the case of most violent volcanic eruptions. Risk is a complex function of the probability of eruptions of various intensities at a given volcano and of the location of the site in question with respect to the volcano.

The most dangerous volcanic phenomena happen very quickly. For instance, the time interval between the beginning of an eruption and the appearance of the first nuées ardentes may be only a matter of hours. Fortunately, such events usually are preceded by visible signs of eruption. No volcanic catastrophes have occurred at the very start of an eruption. Consequently, this affords a certain length of time to take evasive measures. Even so, because less than one out of several hundred eruptions proves dangerous for a neighbouring population, complete evacuation, presuming that an accurate prediction of an event could be made, would be unlikely to take place before an eruption becomes alarming.

Baker (1979) estimated that the active life span of most volcanoes probably is between one and two million years. Since the activity frequently follows a broadly cyclical pattern, some benefit in terms of hazard assessment may accrue from the determination of the recurrence interval of particular types of eruption, the distribution of the resulting deposits, the magnitude of events and the recognition of any short-term patterns of activity. For instance, according to Booth (1979), four categories of hazard have been distinguished in Italy:

1. Very high frequency events with mean recurrence interval, (MRI), of less than two years. The area affected by such events is usually less than 1 km^2.
2. High frequency events with MRI values of 2 to 200 years. In this category, damage may extend up to 10 km^2.
3. Low frequency events with MRI values 200 to 2000 years. Spacial damage may cover over 1000 km^2.
4. Very low frequency events that are associated with the most destructive eruptions and have MRI values in excess of 2000 years. The area affected may be greater than 10,000 km^2.

The return periods of particular types of activity of individual volcanoes or centres of volcanic activity can be obtained by thorough stratigraphical study and dating of the associated deposits so as to reconstruct past events. Used in conjunction with any available historical records, these data form the basis for assessing the degree of risk involved. However, it is unlikely that such studies could ever be refined sufficiently for the time of activity to be predicted to within a decade. Furthermore, because it is unlikely that man will be capable of significantly influencing the degree of hazard, then the reduction of risk can only be achieved by reducing exposure of life and property to volcanic hazard.

Booth (1979) divided volcanic hazards into six categories, namely, premonitory earthquakes, pyroclast falls, pyroclast flows and surges, lava flows, structural collapse, and associated hazards. Each type represents a specific phase of activity during a major eruptive cycle of a polygenetic volcano and may occur singly or in combination with other types. Damage resulting from volcano-seismic activity is rare, although intensities on the Mercalli scale varying from VI to IX have been recorded over limited areas.

Hazards associated with volcanic activity also include destructive floods and mudflows (lahars) caused by sudden melting of snow and ice that cap high volcanoes, by intense downfalls of rain (vast quantities of steam may be given off during an eruption) or by the rapid collapse of a crater lake. Lahars can prove just as destructive as pyroclastic flows and can travel appreciable distances in a matter of minutes. Tsunamis generated by explosive eruptions can prove extremely dangerous. Poisonous gases are a further threat to life. Air blasts, shock waves and counter blasts are relatively minor hazards, although windows may be broken several tens of kilometres away from major eruptions.

Detailed geological mapping of the products of volcanic eruption is required in order to produce maps of volcanic hazard zones (Fig. 8.3). Hazard zoning entails the identification of areas liable to be affected adversely by particular types of volcanic eruption during an episode of activity. Events with a mean recurrence interval of less than 5000 years should be taken into account in the production of maps of volcanic hazard zoning, and data on any events that have taken place in the last 50,000 years are probably significant. Two types of maps are useful for economic and social planning. One indicates areas liable to suffer total destruction by lava flows, nueés ardentes and lahars. The other shows areas likely to be affected temporarily by damaging, but not destructive phenomena including heavy falls of ash, toxic emissions and the pollution of surface or underground waters. Examples of volcanic risk maps include expected ash-fall depths, lava and pyroclastic debris flow paths and the areal extent of lithic missile fallout. Such maps are needed by local and national governments so that appropriate land uses, building codes and civil defence responses can be incorporated into the planning procedures.

Losses caused by volcanic eruptions can be reduced by a combination of prediction, preparedness and land-use control. Emergency measures include alerting the public to

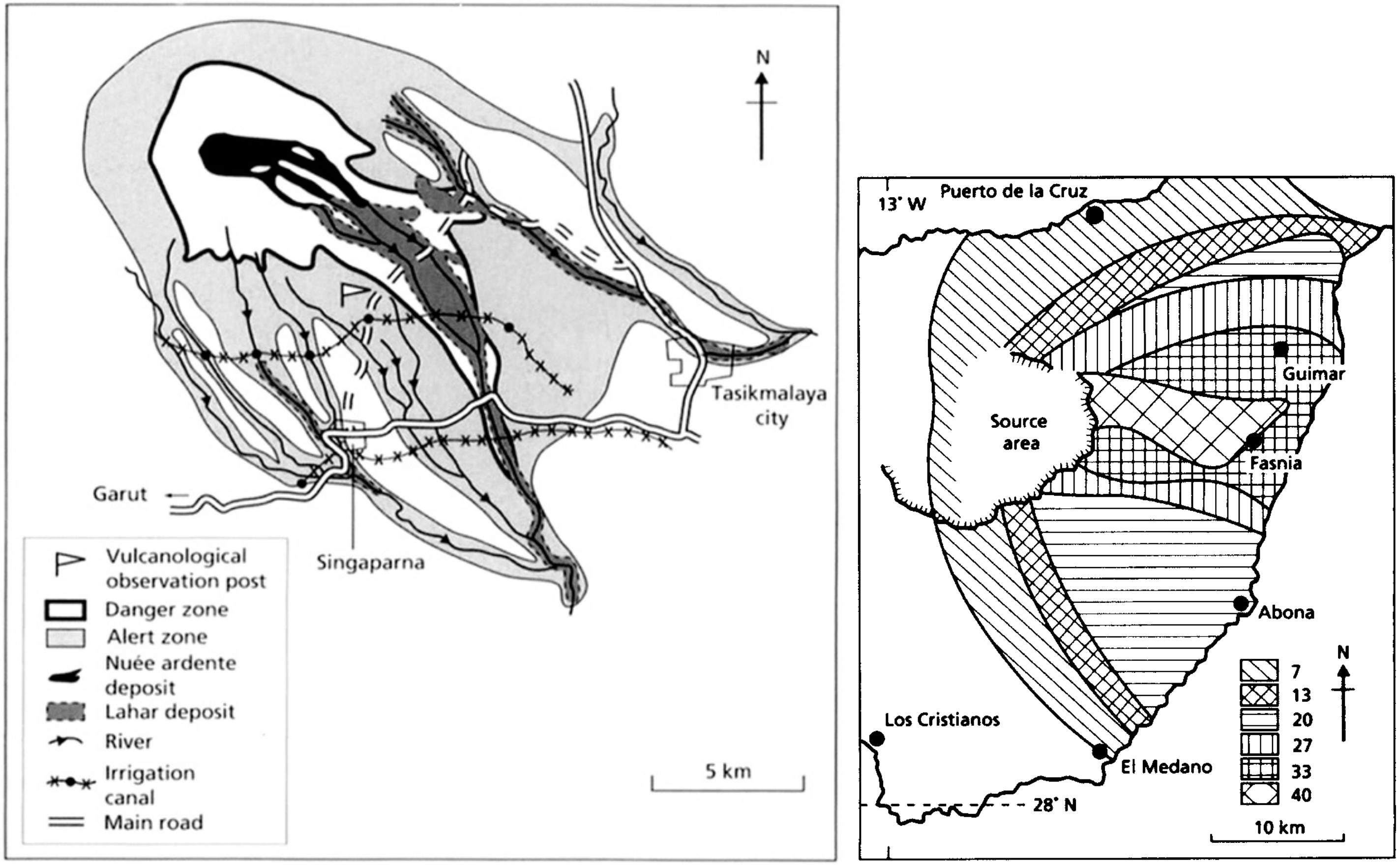


Figure 8.3

(a) Preliminary hazard map of Galunggung volcano, Java, showing the extent of nuee ardente and lahar flow in 1982 (after Suryo and Clarke, 1985). (b) Frequency with which specific areas on Tenerife have been significantly affected by air-fall deposits during the last 50,000 years (after Booth, 1979). With kind permission of the Geological Society.

the hazard, followed by evacuation. Appropriate structural measures for hazard reduction are not numerous, but include building steeply pitched reinforced roofs that are unlikely to be damaged by ash fall, and constructing walls and channels to deflect lava flows. Hazard zoning, insurance, local taxation and evacuation plans are appropriate non-structural measures to put into effect. Risk management depends on identifying hazard zones and forecasting eruptions. The levels of risk must be defined and linked to appropriate social responses. Volcanic zoning is constrained by the fact that the authorities and public are reluctant to make costly adjustments to a hazard that may have a mean recurrence interval of 1000 years or more. Data on recurrence intervals are thus critical to zonation. In the long term, appropriate controls should be placed on land use and the location of settlements.

Earthquakes

Although earthquakes have been reported from all parts of the world, they are primarily associated with the margins of the crustal plates that move with respect to each other. Differential displacements give rise to elastic strains that eventually exceed the strength of the rocks involved and fault movement then occurs. The strained rocks rebound along the fault under the elastic stresses until the strain is partly or wholly dissipated. Initially, movement may occur over a small area of a fault plane, to be followed later by slippage over a much larger surface. Such initial movements account for the foreshocks that precede an earthquake. These are followed by the main movement, but complete stability is not restored immediately. The shift of rock masses involved in faulting relieves the main stress but develops new stresses in adjacent areas. Because stress is not relieved evenly everywhere, minor adjustments may arise along a fault plane and thereby generate aftershocks. The decrease in strength of the aftershocks is irregular, and occasionally, they may continue for a year or more.

The epicentre of an earthquake is located on the surface of the Earth immediately above the focus (i.e. the location of the origin of the earthquake), and shock waves radiate from the focus in all directions. Earthquake foci are confined to within a limited zone of the upper Earth, the lower boundary occurring at around 700 km depth from the surface. No earthquakes are known to have originated below this level. Moreover, earthquakes rarely originate at the Earth's surface. In fact, most earthquakes originate within the upper 25 km of the Earth. Because of its significance, the depth of foci has been used as the basis of a threefold classification of earthquakes; those occurring within the upper 70 km are referred to as shallow, those located between 70 and 300 km as intermediate, and those between 300 and 700 km as deep. Seventy percent of all earthquakes are of shallow type.

An earthquake propagates three types of shock waves. The first pulses that are recorded on a seismograph are the P or compression waves. The next pulses recorded are the S or shear waves. S waves usually have larger amplitude than the P waves but the latter travel almost three times as fast as the former. Both P and S waves travel directly from their source to the surface. The third type of shocks are known as the L waves. These waves travel from the focus of the earthquake to the epicentre above, and from there they radiate over the Earth's surface. P waves are not as destructive as S or L waves. This is because they have smaller amplitude, and the force their customary vertical motion creates rarely exceeds the force of gravity. On the other hand, S waves may develop violent tangential vibrations strong enough to cause great destruction. The intensity of an earthquake depends on the amplitude and frequency of wave motion. S waves commonly have a higher frequency than L waves, nevertheless the latter may be more powerful because of their larger amplitude.

The hazards due to seismicity include the possibility of a structure being severed by fault displacement but a much more likely event is damage due to shaking (Fig. 8.4). The destruction wrought by an earthquake depends on many factors. Of prime importance are the magnitude of the event, its duration and the response of buildings and other elements of the infrastructure. In addition, other hazards such as landslides, floods, subsidence, tsunamis and secondary earthquakes may be triggered by a seismic event (Khazai and Sitar, 2004).

The severest earthquakes wreak destruction over areas of 2500 km^2 or more, however, most only affect tens of square kilometres. The strengths of earthquakes may be expressed in terms of intensity or magnitude. Earthquake intensity scales are a qualitative expression of the damage caused by an event at a given location. The most widely accepted intensity scale is the Mercalli scale given in Table 8.1. The magnitude of an earthquake is an instrumentally measured quantity and is related to the total amount of elastic energy released when overstrained rocks suddenly rebound and generate shock waves. It is expressed on a logarithmic scale. An earthquake of magnitude 2 is the smallest likely to be felt by humans, and earthquakes of magnitude 5 or less are unlikely to cause damage to well-constructed buildings. The maximum magnitude of earthquakes is limited by the amount of strain energy that a rock mass can sustain before failure occurs, hence the largest tremors have had a magnitude of around 8.9. Such an event causes severe damage over a wide area.

The magnitude of an earthquake event depends on the length of the fault break and the amount of displacement that occurs. Generally, movement only occurs on a limited length of a fault during one event. A magnitude 7 earthquake would be produced if a 150 km long fault underwent a displacement of about 1.0 m. The length of the fault break during

Table 8.1. Modified Mercalli Scale (1956 version) with Cancani's equivalent acceleration

Degrees	Description	Acceleration* (mm s^{-2})
I	Not felt. Only detected by seismographs.	Less than 2.5
II	Feeble. Felt by persons at rest, on upper floors, or favourably placed.	2.5 to 5.0
III	Slightly felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as earthquake.	5.0 to 10
IV	Moderate. Hanging objects swing. Vibration like passing of heavy trucks, or sensation or a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV wooden walls and frames creak.	10 to 25
V	Rather strong. Felt outdoors, direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing. Shutters and pictures move. Pendulum clocks stop, start, change rate.	25 to 50
VI	Strong. Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Ornaments, books, etc., fall off shelves. Pictures fall off walls. Furniture moved or overturned. Weak plaster and masonry cracked. Small bells ring (church, school). Trees, bushes shaken visibly or heard to rustle.	50 to 100
VII	Very strong. Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices, also unbraced parapets and architectural ornaments. Some cracks in masonry C. Waves on ponds, water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.	100 to 250
VIII	Destructive. Steering of motor cars affected. Damage to masonry C, partial collapse. Some damage to masonry B, none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down, loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.	250 to 500

IX	Ruinous. General panic. Masonry D destroyed, masonry C heavily damaged, sometimes with complete collapse, masonry B seriously damaged. General damage to foundations. Frame structures, if not bolted, shifted off foundations. Frames cracked, serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas, sand and mud ejected, earthquake fountains, sand craters.	500 to 1000
X	Disastrous. Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dykes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.	1000 to 2500
XI	Very disastrous. Rails bent greatly. Underground pipelines completely out of service.	2500 to 5000
XII	Catastrophic. Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.	Over 5000

Types of masonry used in USA: Masonry A: good workmanship, mortar and design, reinforced laterally and bound together by using steel, concrete, etc., designed to resist lateral forces. Masonry B: good workmanship and mortar, reinforced but not designed to resistance of lateral forces. Masonry C: ordinary workmanship and mortar, no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces. Masonry D: weak materials such as adobe, poor mortar, low standards of workmanship, weak horizontally.

*These are not peak accelerations as instrumentally recorded.

Figure 8.4

Subsidence of Government Hill School, Anchorage, Alaska, after the earthquake of 1964. Soils failed and moved downslope, leaving part of the school on unmoved ground, while dropping the rest into a graben.

a particular earthquake is normally only a fraction of the true length of the fault. Individual fault breaks during simple earthquakes have ranged in length from less than a kilometre to several hundred kilometres. What is more, fault breaks do not only occur in association with large and infrequent earthquakes but also in association with small shocks and continuous slow slippage, known as fault creep. Fault creep may amount to several millimetres per year and progressively deforms buildings located across such faults (Doolin et al., 2005).

There is little information available on the frequency of breaking along active faults. All that can be said is that some master faults have suffered repeated movements; in some cases, it has recurred in less than 100 years. By contrast, much longer intervals, totalling many thousands of years, have occurred between successive breaks. Therefore, because movement has not been recorded in association with a particular fault in an active area, it cannot be concluded that the fault is inactive.

Many seismologists believe that the duration of an earthquake is the most important factor as far as damage or failure of structures, soils and slopes are concerned. Buildings may remain standing in earthquakes that last 30 s, whereas strong motions continuing for 100 s would bring about their collapse. For example, the duration of the main rupture along the fault responsible for the Kobe earthquake in Japan (January 1995; M = 7.2) was 11 s. However, the duration of strong motions was increased in some parts of the city located on soft soils to as long as 100 s (Esper and Tachibana, 1998). What is important in hazard assessment is the prediction of the duration of seismic shaking above a critical ground acceleration threshold. The magnitude of an earthquake affects the duration much more than it affects the maximum acceleration, since the larger the magnitude, the greater the length of the ruptured fault. Hence, the more extended is the area from which seismic waves are emitted. With increasing distance from a fault, the duration of shaking is longer but the intensity of shaking is less, the higher frequency waves being attenuated more than the lower ones.

The physical properties of the soils and rocks through which seismic waves travel, as well as the geological structure, also influence surface ground motion. For example, if a wave traverses vertically through granite overlain by a thick uniform deposit of alluvium, then the amplitude of the wave at the surface should theoretically be double that at the alluvium–granite contact. Maximum acceleration within an earthquake source area may exceed 2 g for competent bedrock. On the other hand, normally consolidated clays with low plasticity are incapable of transmitting accelerations greater than 0.1 to 0.15 g to the surface. Clays with high plasticity allow accelerations of 0.25 to 0.35 g to pass through. Saturated sandy clays and medium dense sands may transmit 50 to 60% g, and in clean gravel and dry dense sand, accelerations may reach much higher values.

The response of structures on different foundation materials has proved surprisingly varied. In general, structures not specifically designed for earthquake loadings have fared far worse on soft saturated alluvium than on hard rock. This is because motions and accelerations are much greater in deep alluvium than in rock. By contrast, a rigid building may suffer less on alluvium than on rock. The explanation is attributable to the alluvium having a cushioning effect, and the motion may be changed to a gentle rocking. This is easier on such a building than the direct effect of earthquake motions experienced on harder ground. Nonetheless, alluvial ground beneath any kind of poorly constructed structure facilitates its destruction.

Ground vibrations often lead to the consolidation of sandy soils and associated settlement of the ground surface. Loosely packed saturated sands and silts tend to liquify, thus losing their bearing capacity (Fig. 8.5). Bray et al. (2004) described the liquefaction of soils after the Koccaeli earthquake at Adapzari, Turkey, in 1999, when hundreds of structures settled, slid, tilted and collapsed. If liquefaction occurs in a sloping soil mass, the entire mass begins to move as a flow slide. Such slides develop in loose saturated silts and sands during earthquakes.

Figure 8.5

Collapsed building in the Marina District of San Francisco, October 1989, as a result of the Loma Prieta earthquake. The ground floor of this three-storey building failed, and the second floor collapsed when the silty ground was liquefied, leaving only the third storey.

Loose saturated silts and sands often occur as thin layers underlying firmer materials. In such instances, liquefaction of the silt or sand during an earthquake may cause the overlying material to slide over the liquefied layer. Structures on the main slide are frequently moved without suffering damage. However, a graben-like feature often forms at the head of the slide, and buildings located in this area are subjected to large differential settlements and are often destroyed. Buildings near the toe of the slide are commonly heaved upwards or are pushed over by the lateral thrust.

Clay soils do not undergo liquefaction when subjected to earthquake activity. However, large deformations can develop under repeated cycles of loading, although the peak strength remains about the same. Nonetheless, these deformations can reach the point where, for all practical purposes, the soil has failed. Major slides can result from failure in deposits of clay.

Details about the occurrence, magnitude and effects of earthquakes through time can be obtained by carrying out long-term seismic monitoring. In the absence of suitable instrumental data, the seismic history of an area may be established by interpretation of historical, archaeological and geological evidence.

In earthquake prone areas, any decision making relating to urban and regional planning, or for earthquake resistant design, must be based on information concerning the characteristics of probable future earthquakes. However, the lack of reliable data to be used for design purposes has been a cause for concern in engineering seismology. Hence, the engineer may on occasions accept an element of risk above what would otherwise be considered normal. This risk, under certain conditions, may prove economically acceptable for structures of relatively short life. However, this is not acceptable for certain structures whose failure or damage during an earthquake may lead to disaster. A considerable amount of informed judgement and technical evaluation is needed to determine what is an acceptable risk for a given project, and caution is required in the selection of earthquake design parameters. There are a number of methods for selecting appropriate earthquake design parameters, which may involve estimates of strong ground motions, but all of them depend on the quality of the input data. The estimation of the maximum probable earthquake ground motion, and when and where it will occur is extremely difficult or impossible to assess.

Seismic zoning and micro-zoning provide a means of regional and local planning in relation to the reduction of seismic hazard, as well as being used in terms of earthquake-resistant design. While seismic zoning takes into account the distribution of earthquake hazard within a region, seismic micro-zoning defines the distribution of earthquake risk in each seismic zone. Seismic evidence obtained instrumentally and from the historical record can be used to produce maps of seismic zoning (Fig. 8.6). Maximum hazard levels can be based on the assumption that future earthquakes will occur with the same maximum magnitudes and intensities recorded at a given location as in the past. Hence, seismic zoning provides a broad picture of the earthquake hazard that can occur in seismic regions and so has led to a reduction of earthquake risk. Detailed seismic micro-zoning maps should take account of local geological characteristics, as well as the differences in the spectrum of seismic vibrations and the probability of the occurrence of earthquakes of various intensities.

Most studies of the distribution of damage attributable to earthquakes indicate that areas of severe damage are highly localized and that the degree of damage can change abruptly over short distances. These differences are frequently due to changes in soil conditions or local geology. Such behaviour has an important bearing on seismic micro-zoning. Hence, seismic micro-zoning maps can be used for detailed land-use planning and for insurance risk evaluation. These maps obviously are most detailed and more accurate where earthquakes have occurred quite frequently and the local variations in intensity have been recorded. A map of micro-seismic zoning for Los Angeles County is shown in Figure 8.7. It distinguishes three zones of ground response, active faults likely to give rise to ground rupture and areas of potential landslide and liquefaction risk.

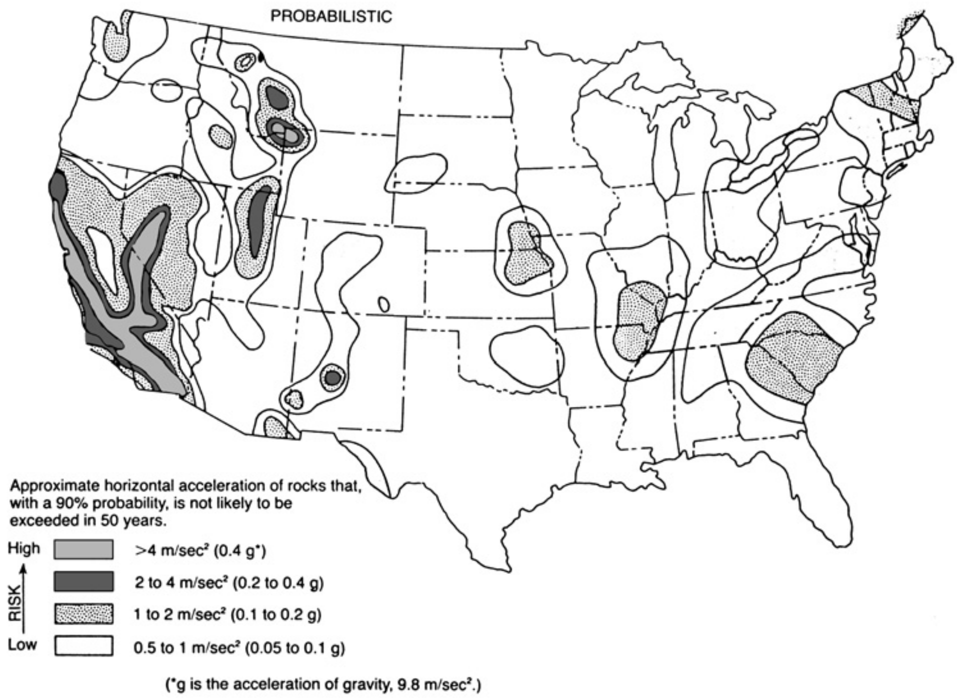


Figure 8.6

Probabilistic representation of seismic hazard in the United States. Shaded areas give a probabilistic horizontal acceleration of the ground that, with a 90% probability, is not likely to be exceeded in 50 years (Algermissen and Perkins, 1976).

Landslides and slope movements

Landslide hazard refers to the probalility of a landslide of a given size occurring within a specified period of time within a particular area. Most landslides occur in areas previously affected by instability, and few occur without prior warning. Hence, it is important to carry out careful surveys of areas that appear potentially unstable. The associated risk relates to the loss of lives or damage to property. Consequently, landslide hazard has to be assessed before landslide risk can be estimated (Ko Ko et al., 2003).

Generally, the purpose of mapping landslide hazards is to locate problem areas and to help understand why, when and where landslides are likely to occur. Most landslide investigations are local and site specific in character, being concerned with establishing the nature and degree of stability of a certain slope or slope failure or all of these (Whitworth et al., 2005). Landslides should be classified as active or inactive, historical data being of importance, as well as recent evidence of movement and freshness of form (Hack et al., 2003). Such investigations involve desk studies; geomorphological mapping from aerial photographs or

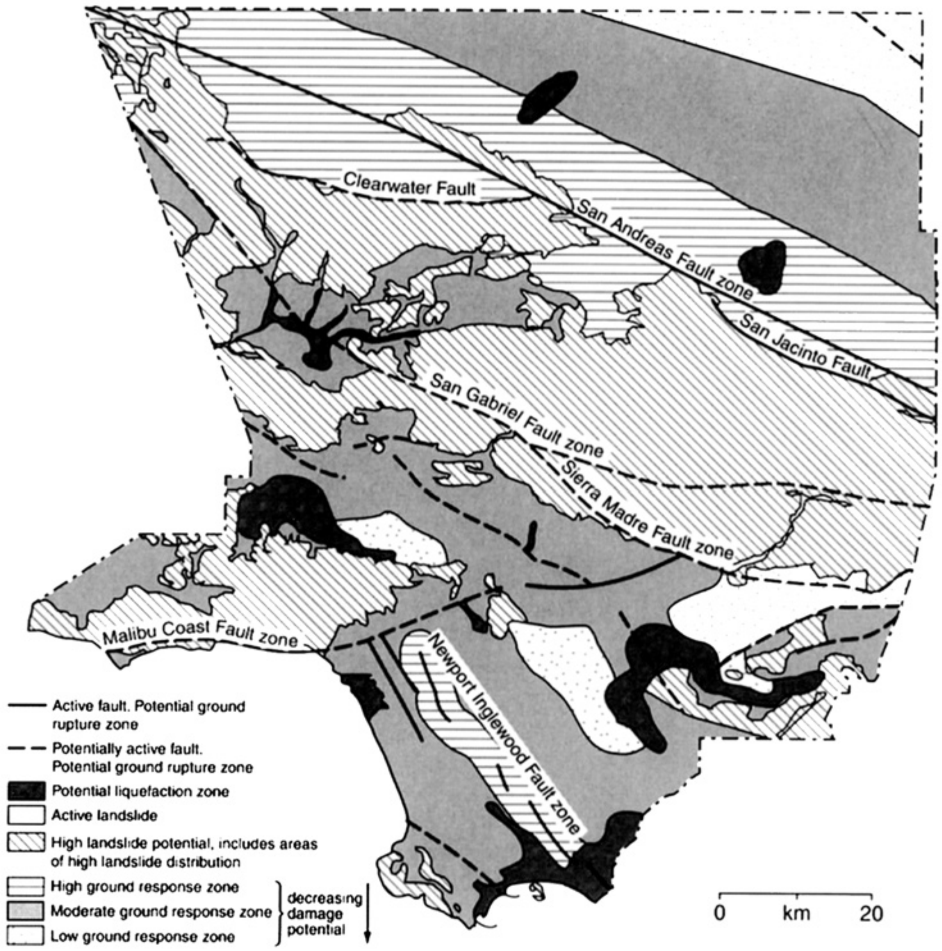


Figure 8.7

Microseismic zoning map of Los Angeles County (courtesy of Los Angeles County Planning Department).

satellite imagery, as well as mapping in the field; subsurface investigation by trenches, pits or boreholes; sampling and testing, especially of slip surface material; monitoring of surface movements; monitoring pore water pressures with piezometers; and analysis of the data obtained. Digitization of data from aerial photographs or remote sensing imagery enables this to be handled in geographical information systems through which the landslide controlling factors can be evaluated. There are a number of data sources from which background information may be obtained. These include reports, records, papers, topographical and geological maps, and aerial photographs and remote sensing imagery. Field investigation, monitoring, sampling and laboratory testing may provide more accurate and therefore more valuable data (Vandwater et al., 2005). Many landslide maps only show the hazards known at a particular time, whereas others provide an indication of the possibility of landslide occurrence (Fig. 8.8). The latter are landslide susceptibility maps and involve some estimation of relative risk (Van Westen et al., 2003).

Effective landslide hazard management has done much to reduce economic and social losses due to slope failure by avoiding the hazards or by reducing the damage potential (Schuster, 1992). This has been accomplished by prohibition, restrictions or regulations placed on development in landslide-prone areas; application of excavation grading, landscaping and construction codes; use of remedial measures to prevent or control slope failures; and landslide warning systems. These measures may be used in a variety of combinations to help solve both existing and potential landslide problems. They are generally applicable to flows, slides and falls. Recurrent damage from landslides can be avoided by permanently evacuating areas that continue to experience slope failures. Structures may be removed or converted to a use that is less vulnerable to landslide damage. Various types of land-use and land-development regulations can be used to reduce landslide hazards. They are often the most economical and most effective means available to a local government. Landslide-prone areas can be used as open spaces or the density of development can be kept to a minimum to reduce the potential for damage. Zoning and subdivision regulations, as well as moratoriums on rebuilding, can be used to meet these objectives.

As the same preventative or corrective work cannot always be applied to different types of slides, it is important to identify the type of slide that is likely to take place or that has taken place. Also, it is important to bear in mind that landslides may change in character and that they are usually complex. When it comes to the correction of a landslide, as opposed to its prevention, since the limits and extent of the slide are generally well defined, the seriousness of the problem can be assessed. In such instances, consideration must be given to the stability of the area immediately adjoining the slide. Obviously, any corrective treatment must not adversely affect the stability of the area about the slide.

If landslides are to be prevented, then areas of potential landsliding must be identified, as must their type and possible amount of movement. Then, if the hazard is sufficiently real, the engineer can devise a method of preventative treatment. Economic considerations, however, cannot be disregarded. In this respect, it is seldom economical to design cut slopes sufficiently flat to preclude the possibility of landslides.

Landslide prevention may be brought about by reducing the activating forces, by increasing the forces resisting movement or by avoiding or eliminating the slide. Reduction of the activating forces can be accomplished by removing material from that part of the slide that provides the force that will give rise to movement. Complete excavation of potentially unstable material from a slope may be feasible; however, there is an upper limit to the amount of material that can be removed economically. Although partial removal is suitable for dealing with

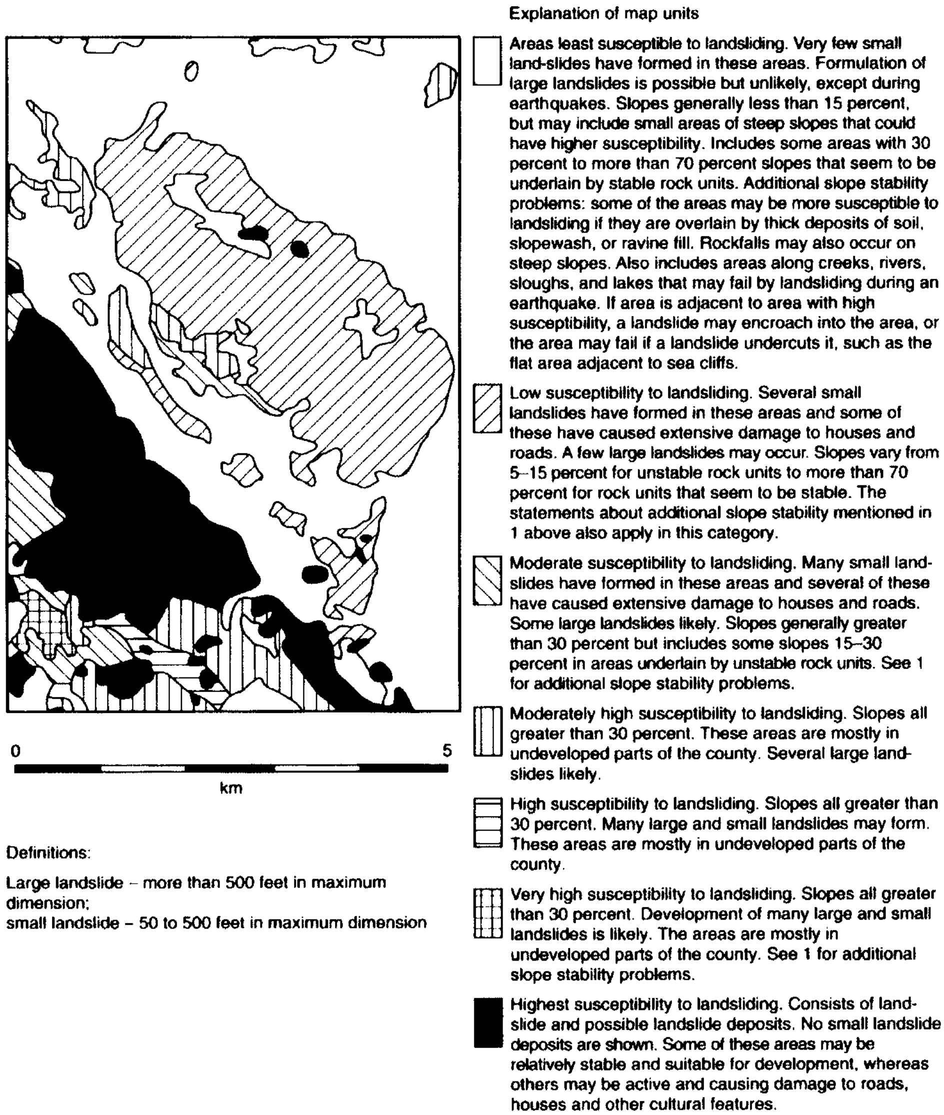


Figure 8.8

Landslide susceptibility classification used in the study of San Mateo County, California (after Brabb et al., 1972). With the permission of the United States Geological Survey.

most types of mass movement, it is inappropriate for some types. For example, removal of head has little influence on flows or slab slides. On the other hand, this treatment is suitable for rotational slips. Slope flattening, however, is rarely applicable to rotational or slab slides. Slope reduction may be necessary in order to stabilize the toe of a slope and thus prevent successive undermining with consequent spread of failure upslope. Benching can be used on steeper slopes. It brings about stability by dividing a slope into segments.

Drainage is the most generally applicable preventative and corrective treatment for slides, regardless of type. Indeed, drainage is the only economic way of dealing with landslides involving the movement of several million cubic metres of material. The surface of a landslide is generally uneven, hummocky and traversed by deep fissures. This is particularly the case when a slipped area consists of a number of slices. Water collects in depressions and fissures, and pools and boggy areas are formed. In such cases, the first remedial measure to be carried out is surface drainage (see Chapter 9).

Measures to control rockfall include catch fences, benches, wire mesh and rock traps. Restraining structures control landslides by increasing the resistance to movement. They include, for example, retaining walls, reinforced earth structures, shotcrete, cribs, gabions, buttresses, piling, dowels, rock bolts and rock anchors (see Chapter 9). The following minimum information is required to determine the type and size of a restraining structure:

1. The boundaries and depth of the unstable area, its moisture content and its relative stability
2. The type of slide that is likely to develop or has occurred
3. The foundation conditions since restraining structures require a satisfactory anchorage

River Action and Flooding

All rivers form part of a drainage system, the form of which is influenced by rock type and structure, the nature of the vegetation cover and the climate. An understanding of the processes that underlie river development forms the basis of proper river management.

Rivers form part of the hydrological cycle in that they carry run-off from precipitation. This run-off is the surface water that remains after evapotranspiration and infiltration into the ground have taken place. Some precipitation may be frozen, only to contribute to run-off at some later time, while any precipitation that has soaked into the ground may reappear as springs where the water table meets the ground surface. Although, due to intense rainfall or in areas with few channels, the run-off may occur as a sheet, generally speaking it becomes concentrated into channels. These become eroded by the flow of water so that they form valleys (see Chapter 3).

Floods represent one of the commonest types of geological hazards (Fig. 8.9). However, the likelihood of flooding is more predictable than some other types of hazards such as earthquakes, volcanic eruptions and landslides. Most disastrous floods are the results of excessive precipitation or snowmelt, that is, they are due to excessive surface run-off. It usually takes some time to accumulate enough run-off to cause a major disaster. This lag time is an important parameter in flood forecasting. Flash floods prove the exception. In most regions, floods occur more frequently in certain seasons than others. As the volume of water in a river is increased during times of flood, its erosive power increases accordingly. Hence, the river carries a much higher sediment load. Deposition of the latter where it is not wanted also represents a problem.

The influence of human activity can bring about changes in drainage basin characteristics, for example, removal of forest from parts of a river basin can lead to higher peak discharges that

Figure 8.9

Collapse of John Ross Bridge due to flooding of the Tugela River, Natal, South Africa, September 1987.

generate increased flood hazards. A notable increase in flood hazard can arise as a result of urbanization, the impervious surfaces created mean that infiltration is reduced, and together with storm water drains, they give rise to increased run-off. Not only does this produce higher discharges, but lag-times also are reduced. The problem of flooding is acute where rapid expansion has led to the development of urban sprawl without proper planning, or worse, where informal settlements have sprung up.

A flood can be defined in terms of height or stage of water above some given point such as the banks of a river channel. However, rivers generally are considered in flood when the level has risen to an extent that damage occurs. The discharge rate provides the basis for most methods of predicting the magnitude of flooding, the most important factor being the peak discharge that is responsible for maximum inundation. Not only does the size of a flood determine the depth and area of inundation but it primarily determines the duration of a flood. These three parameters influence the velocity of flow of flood waters, all four being responsible for the damage potential of a flood. The physical characteristics of a river basin together with those of the river channel affect the rate at which discharge downstream occurs. The average time between a rainstorm event and the consequent increase in river flow is referred to as the lag-time. Lag-time can be measured from the commencement of rainfall to the peak discharge or from the time when actual flood conditions have been attained (e.g. bankfull discharge) to the peak discharge. This lag-time is an important parameter in flood forecasting. Calculation of the lag-time, however, is a complicated matter. Nonetheless, once enough data on rainfall and run-off versus time have been obtained and analysed, an estimate of where and when flooding will occur along a river system can be made.

Analyses of discharge are related to the recurrence interval to produce a flood frequency curve (Fig. 8.10). The recurrence interval is the period of years within which a flood of a given

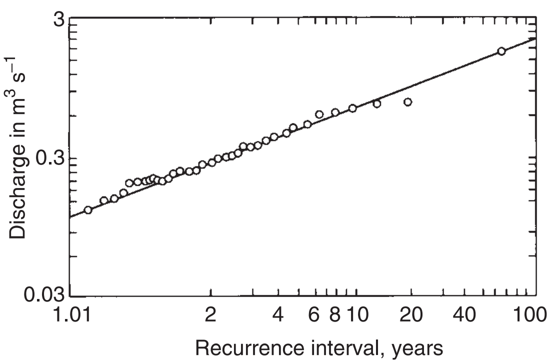


Figure 8.10

A flood frequency curve.

magnitude or greater occurs and is determined from field measurements at a gauging centre. It applies to that station only. The flood frequency curve can be used to determine the probability of the size of discharge that could be expected during any given time interval.

The lower stage of a river, in particular, can be divided into a series of hazard zones based on flood stages and risk. A number of factors have to be taken into account when evaluating flood hazard, such as the loss of life and property, erosion and structural damage; disruption of socio-economic activity including transport and communications; contamination of water, food and other materials; and damage of agricultural land and loss of livestock. A floodplain management plan involves the determination of such zones. These are based on the historical evidence related to flooding that include the magnitude of each flood and the elevation it reached, as well as the recurrence intervals; the amount of damage involved; the effects of urbanization and any further development; and an engineering assessment of flood potential. Maps then are produced from such investigations that, for example, show the zones of most frequent flooding and the elevation of the flood waters. Kenny (1990) recognized four geomorphological flood hazard map units in central Arizona, which formed the basis of a flood management plan (Table 8.2).

Flood plain zones can be designated for specific types of land-use. For instance, in the channel zone, water should be allowed to flow freely without obstruction, for example, bridges should allow sufficient waterway capacity. Another zone could consist of that area with recurrence intervals of 1 to 20 years that could be used for agricultural and recreational purposes. Buildings would be allowed in the zone encompassing the 20 to 100 year recurrence interval, but they would have to have some form of protection against flooding. However, a line that is drawn on a map to demarcate a floodplain zone may encourage a false sense of security and, as a consequence, development in the upslope area may be greater than it otherwise would. At some point in time, this line is likely to be transgressed, causing more damage than would have been the case without a floodplain boundary being so defined.

Land-use regulation seeks to obtain the beneficial use of floodplains with minimum flood damage and minimum expenditure on flood protection. In other words, the purpose of land-use regulation is to maintain an adequate floodway (i.e. the channel and those adjacent areas of the floodplain that are necessary for the passage of a given flood discharge) and to regulate land-use development alongside it.

Reafforestation of slopes denuded of woodland tends to reduce run-off and thereby lowers the intensity of flooding. As a consequence, forests are frequently used as a watershed management technique. They are most effective in relation to small floods where the possibility exists of reducing flood volumes and delaying flood response. Nonetheless, if the soil is saturated, differences in interception and soil moisture storage capacity due to forest cover will be ineffective in terms of flood response.

Table 8.2. Generalized flood hazard zones and management strategies (after Kenny, 1990). With the kind permission of the Geological Society

Flood hazard zone I (active floodplain area) Prohibit development (business and residential) within floodplain. Maintain area in a natural state as an open area or for recreational uses only
Flood hazard zone II (Alluvial fans and plains with channels less than a metre deep, bifurcating and intricately interconnected systems subject to inundation from overbank flooding) Flood proofing to reduce or prevent loss to structures is highly recommended. Residential development densities should be relatively low; development in obvious drainage channels should be prohibited. Dry stream channels should be maintained in a natural state and/or the density of native vegetation should be increased to facilitate superior water drainage retention and infiltration capabilities. Installation of upstream stormwater retention basins to reduce peak water discharges. Construction should be at the highest local elevation site where possible
Flood hazard zone III (Dissected upland and lowland slopes; drainage channels where both erosional and depositional processes are operative along gradients generally less than 5%) Similar to flood hazard zone II Roadways that traverse channels should be reinforced to withstand the erosive power of a channelled stream flow.
Flood hazard zone IV (Steep gradient drainages consisting of incised channels adjacent to outcrops and mountain fronts characterized by relatively coarse bedload material) Bridges, roads and culverts should be designed to allow unrestricted flow of boulders and debris up to a metre or more in diameter. Abandon roadways that presently occupy the wash flood plains. Restrict residential dwelling to relatively level building sites. Provisions for subsurface and surface drainage on residential sites should be required. Stormwater retention basins in relatively confined upstream channels to mitigate against high peak discharges.

Emergency action involves erection of temporary flood defences and possible evacuation. A flood warning system can be used to alert a community to the danger of flooding. The success of such measures depends on the ability to predict floods and the effectiveness of the warning system. However, widespread use of flood warning is usually only available in highly sensitive areas. Warning systems often work well in large catchment areas that allow enough time between the rainfall or snowmelt event and the resultant flood peak to allow evacuation and any other measures to be put into effect. By contrast, in small tributary areas, especially those with steep slopes or appreciable urban development, the lag-time may be so short that, although prompt action may save lives, it is seldom possible to remove or protect property.

No structure of any importance, either in or adjacent to a river, should ever be built without giving due consideration to the damage it may cause by its influence on flood waters or the damage to which it may be subjected by those same waters. The maximum flood that any such structure can safely pass is called the design flood. If a flood of a given magnitude occurs on average once in 100 years, then there is a 1% chance that such a flood will occur during any one year. The important factor to be determined for any design flood is not simply its magnitude but the probability of its occurrence. In other words, is the structure safe against the 2%, 1% or the 0.1% chance flood or against the maximum flood that may ever be anticipated? Once this has been answered, the magnitude of the flood that may be expected to occur with that particular average frequency has to be determined.

River control refers to projects designed to hasten the run-off of flood waters or to confine them within restricted limits, to improve drainage of adjacent lands, to check stream bank erosion or to provide deeper water for navigation. What has to be borne in mind, however, is that a river in an alluvial channel is changing its position continually due to the hydraulic forces acting on its banks and bed. As a consequence, any major modifications to the river regime that are imposed without consideration of the channel will give rise to a prolonged and costly struggle to maintain the change.

Artificial strengthening and heightening of levees or constructing artificial levees are frequent measures employed as protection against flooding. Because these confine a river to its channel, its efficiency is increased. The efficiency of a river is also increased by cutting through the constricted loops of meanders. River water can be run off into the cut-off channels when high floods occur. Canalization or straightening of a river can help to regulate flood flow, and improves the river for navigation (Fig. 8.11). However, canalization in some parts of the world has only proved a temporary expedient. This is because canalization steepens the channel, as sinuosity is reduced. This, in turn, increases the velocity of flow and therefore the potential for erosion. Moreover, the base level for the upper reaches of the river has, in effect, been lowered, which means that channel incision will begin there. Although the flood and drainage problems are temporarily solved, the increase in erosion upstream increases sediment load. This then is deposited in the canalized sections that can lead to a return of the original flood and drainage problems.

Diversion is another method used to control flooding. This involves opening a new exit for part of the river water. But any diversion must be designed in such a way that it does not cause excessive deposition in the main channel, otherwise it defeats its own purpose. If in some localities, little damage will be done by flooding, then these areas can be inundated during times of high flood and so act as safety valves.

Channel regulation can be brought about by training walls or dams that are used to deflect a river into a more desirable alignment or to confine it to lesser widths. Walls and dams can be used to close secondary channels and thus divert or concentrate a river into a preferred course. In some cases, ground sills or weirs need to be constructed to prevent undesirable deepening of the bed by erosion. Bank revetment by pavement, rip-rap or protective

Figure 8.11

Straightening of a small section of the River Nene in Cambridgeshire, England. The river is also being deepened and the banks protected by the installation of steel sheet piles.

mattresses to retard erosion usually is carried out along with channel regulation, or it may be undertaken independently for the protection of the land bordering the river (Fig. 8.12).

Control of the higher stretches of a river in those regions prone to soil removal by gullying and sheet erosion is very important. The removal of the soil mantle means that run-off becomes increasingly more rapid and consequently the problem of flooding is aggravated. These problems were tackled in the Tennessee Valley, United States, by establishing a well-planned system of agriculture so that soil fertility was maintained, and the valley slopes were reafforested. The control of surface water was thereby made more effective. Gullies were filled and small dams were constructed across the headstreams of valleys to regulate run-off. Larger dams were erected across tributary streams to form catchment basins for flood waters. Finally, large dams were built across the main river to smooth flood flow.

Nonetheless, the construction of a dam across a river can lead to other problems. It decreases the peak discharge and reduces the quantity of bedload through the river channel. The width of a river may be reduced downstream of a dam as a response to the decrease in flood peaks. Moreover, scour normally occurs immediately downstream of a dam. Removal of the finer fractions of the bed material by scouring action may cause

Figure 8.12

Revetment composed of fascines of willow being placed along the River Ouse, south of King's Lynn, Lincolnshire, England.

armouring of the channel. Dams can only be effective when the reservoir they impound is not full. Reservoirs for flood control should be so operated that the capacity required for storing flood water is available when needed (Bell and Mason, 1997).

River channels may be improved for navigation purposes by dredging. When a river is dredged, its floor should not be lowered so much that the water level is lowered appreciably. In addition, the nature of the materials occupying the floor and the banks should be investigated. This provides data from which the stability of the slopes of the projected new channel can be estimated and indicates whether blasting is necessary. The rate at which sedimentation takes place provides some indication of the regularity at which dredging should be carried out.

Marine Action

Waves, acting on beach material, are a varying force. They vary with time and place due, first, to changes in wind force and direction over a wide area of sea, and second, to changes in coastal aspects and offshore relief. This variability means that the beach is rarely in equilibrium with the waves. Such a more or less constant state of disequilibrium occurs most frequently where the tidal range is considerable, as waves are continually acting at a different level on the beach.

The rate at which coastal erosion proceeds is influenced by the nature of the coast itself and is most rapid where the sea attacks soft unconsolidated sediments (Fig. 8.13). When soft deposits are being actively eroded, the cliff displays signs of landsliding together with evidence of scouring at its base. For erosion to continue, the debris produced must be removed by the sea. This usually is accomplished by longshore drift. If, on the other hand, material is deposited to form extensive beaches and the detritus is reduced to a minimum size, then the submarine slope becomes very wide. Wave energy is dissipated as the water moves over such beaches and cliff erosion ceases. A methodology for assessing the rate of recession of coastal cliffs has been provided by Lee (2005).

Dunes are formed by onshore winds carrying sand-sized material landward from the beach along low-lying stretches of coast where there is an abundance of sand on the foreshore.

Figure 8.13

Destruction of houses by marine erosion of glacial deposits along the coast of Holderness, East Yorkshire, England.

Dunes act as barriers, energy dissipators and sand reservoirs during storm conditions. Because dunes provide a natural defence against erosion, once they are breached the ensuing coastal changes may be long-lasting. In spite of the fact that dunes inhibit erosion, without beach nourishment they cannot be relied upon to provide protection in the long term along rapidly eroding shorelines. Beach nourishment widens the beach and maintains the proper functioning of the beach-dune system during normal and storm conditions.

The amount of longshore drift along a coast is influenced by coastal outline and wavelength. Short waves can approach the shore at a considerable angle, and generate consistent down-drift currents. This is particularly the case on straight or gently curving shores and can result in serious erosion where the supply of beach material reaching the coast from updrift areas is inadequate. Conversely, long waves suffer appreciable refraction before they reach the coast.

Before any project or beach planning and management scheme can be started, a complete study of the beach must be made. The preliminary investigation of the area concerned should first consider the landforms and rock formations along the beach and adjacent rivers, giving particular attention to their durability and stability. In addition, consideration must be given to the width, slope, composition and state of accretion or erosion of the beach; the presence of bluffs, dunes, marshy areas or vegetation in the backshore area; and the presence of beach structures such as groynes. Estimates of the rates of erosion and the proportion and size of eroded material contributed to the beach must be made, as well as whether these are influenced by seasonal effects (Martinez et al., 1990). For example, Rosser et al. (2005) described the use of terrestrial laser scanning to monitor the rate of coastal erosion of cliffs formed in hard rocks in North Yorkshire, England. The behaviour when weathered of any unconsolidated materials that form the cliffs, together with their slope stability and likelihood of sliding, has to be taken into account. Samples of the beach and underwater material have to be collected and analysed for such factors as their particle size distribution and mineral content. Mechanical analyses may prove useful in helping to determine the amount of material that is likely to remain on the beach, for beach sand is seldom finer than 0.1 mm in diameter. The amount of material moving along the shore must be investigated inasmuch as the effectiveness of the structures erected may depend on the quantity of drift available.

Topographic and hydrographic surveys of an area allow the compilation of maps and charts from which a study of the changes along a coast may be made. Observations should be taken of winds, waves and currents, and information gathered on streams that enter the sea in or near the area concerned. Inlets across a beach need particular evaluation. During normal times, there may be relatively little longshore drift but if upbeach breakthroughs occur in a bar off the inlet mouth, then sand is moved downbeach and is subjected to longshore drift. The contributions made by large streams may vary. For example, material brought down by large floods may cause a temporary, but nevertheless appreciable, increase in the beach width around the mouths of rivers. The effects of any likely changes in sea level also must be taken into account (Clayton, 1990). The data collected can be used to plan a system of coastal defences that also should incorporate a scheme of coastal management (French, 2001).

The groyne is the most important structure used to stabilize or increase the width of the beach by arresting longshore drift (Fig. 8.14). Consequently, they are constructed at right angles to the shore. Groynes should be approximately 50% longer than the beach on which they are erected. Standard types usually slope at about the same angle as the beach. Permeable groynes have openings that increase in size seawards and thereby allow some drift material to pass through them. The common spacing rule for groynes is to arrange them at intervals of one to three groyne lengths. The selection of the type of groyne and its spacing depends on the direction and strength of the prevailing or storm waves, the amount and direction of longshore drift and the relative exposure of the shore. With abundant longshore drift and relatively mild storm conditions, almost any type of groyne appears satisfactory, whilst when the longshore drift is lean, the choice is much more difficult. Groynes, however, reduce the amount of material passing downdrift and can therefore prove detrimental to those areas of the coastline. Their effect on the whole coastal system therefore should be considered.

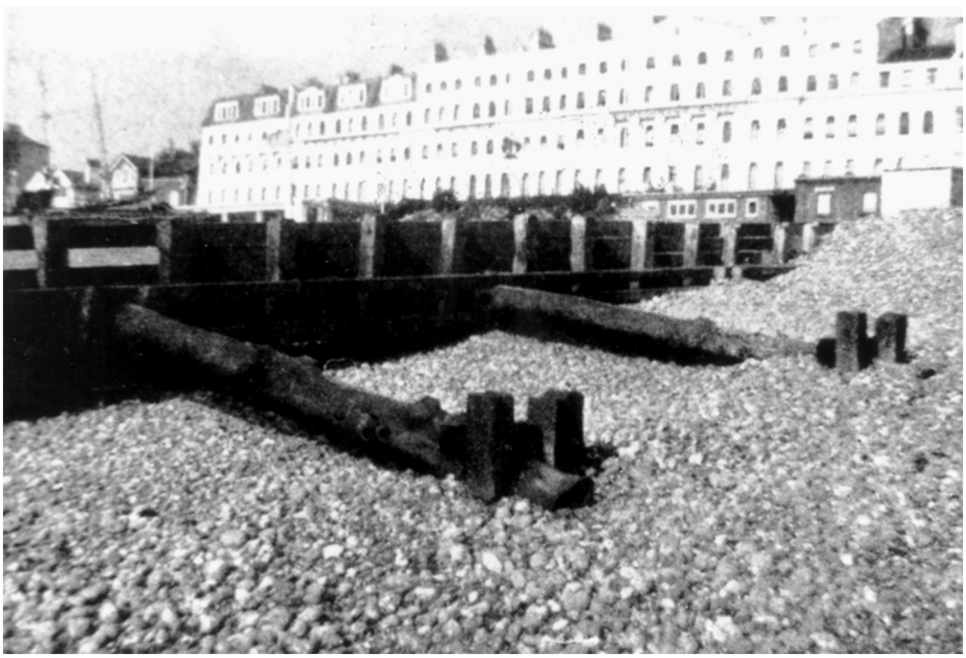


Figure 8.14

Groynes at Eastbourne, south coast of England.

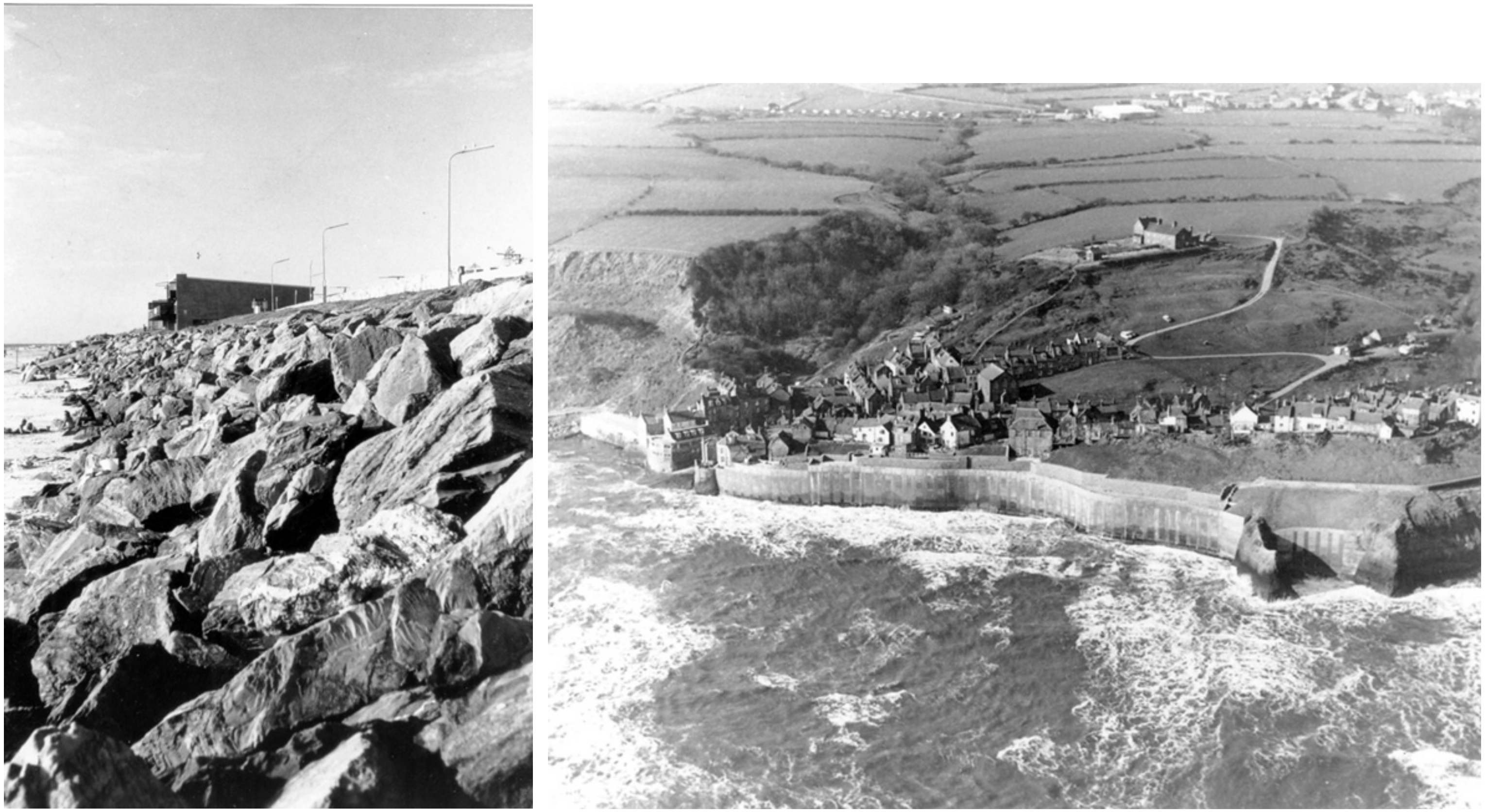
Artificial replenishment of the shore by building beach fills is used either if it is economically preferable or if artificial barriers fail to defend the shore adequately from erosion. In fact, beach nourishment represents the only form of coastal protection that does not adversely affect other sectors of the coast. Unfortunately, it is often difficult to predict how frequently a beach should be renourished. Ideally, the beach fill used for renourishment should have a similar particle size distribution as the natural beach material.

Seawalls and bulkheads are protective waterfront structures. The former range from a simple rip-rap deposit to a regular masonry retaining wall (Fig. 8.15). Bulkheads are vertical walls either of timber boards or of steel sheet piling. Foundation ground conditions for these retaining structures must be given careful attention, and due consideration must be given to the likelihood of scour occurring at the foot of the wall and to changes in beach conditions. Because walls are impermeable, they can increase the backwash and therefore its erosive capability.

Offshore breakwaters and jetties are designed to protect inlets and harbours. Breakwaters disperse the waves of heavy seas, whereas jetties impound longshore drift material up-beach of the inlet and thereby prevent sanding of the channel they protect. Offshore breakwaters commonly run parallel to the shore or at slight angles to it, chosen with respect to the direction that storm waves approach the coast (Fig. 8.16). Although long offshore breakwaters shelter their leeside, they also cause wave refraction and so may generate currents in opposite directions along the shore towards the centre of the sheltered area with resultant impounding of sand. Jetties are usually built at right angles to the shore, although their outer segments may be set at an angle. Two parallel jetties may extend from each side of a river for some distance out to sea and, because of its confinement, the velocity of river flow is increased, which in turn lessens the amount of deposition that takes place. Like groynes, such structures inhibit the downdrift movement of material, and the deprivation of downdrift beaches of sediment may result in serious erosion.

Obviously, prediction of the magnitude of a storm surge ahead of its arrival can help save lives. Storm tide warning services have been developed in various parts of the world. Warnings are usually based on comparisons between predicted and observed levels at a network of tidal gauges where coastal surges tend to move progressively along the coast. To a large extent, the adequacy of a coastal flood forecasting and warning system depends on the accuracy with which the path of the storm responsible can be determined. Storms can be tracked by satellite, and satellite and ground data can be used for forecasting.

Protective measures, such as noted above, are not likely to be totally effective against every storm surge. Nevertheless, flood protection structures are used extensively and often prove



(a) (b)

Figure 8.15

(a) Rip-rap used as revetment, Freemantle, Western Australia, (b) A sea wall at Runswick Bay, North Yorkshire, England.

Figure 8.16

A breakwater formed of tetrapods near Port Elizabeth, South Africa.

substantially effective. Sand dunes and high beaches offer natural protection against floods and can be maintained by artificial nourishment with sand. Dunes can also be constructed by tipping and bulldozing sand or by aiding the deposition of sand by the use of screens and fences. Coastal embankments are usually constructed of local material. Earth embankments are susceptible to erosion by wave action and if overtopped can be scoured on the landward side. Slip failures may also occur. Sea walls may be constructed of concrete or sometimes of steel sheet piling. Permeable sea walls may be formed of rip-rap or concrete tripods. The roughness of their surfaces reduces swash height and the scouring effect of backwash on the foreshore. Barrages may be used to shorten the coastline along highly indented coastlines and at major estuaries. Although expensive, barrages can serve more than one function, not just flood protection. They may produce hydroelectricity, carry communications and freshwater lakes can be created behind them. The Delta Scheme in the Netherlands provides one of the best examples. This was a consequence of the disastrous flooding that resulted from the storm surge of February 1953. Estuaries that have major ports require protection against sea flooding. The use of embankments may be out of the question in urbanized areas as space is not available without expensive compulsory purchase of waterfront land and barrages with locks would impede sea-going traffic. In such situations, movable flood barriers may be used that can be emplaced when storm surges threaten. The Thames barrier in London offers an example (Fig. 8.17).

Figure 8.17

The Thames barrier, London, England.

In the open ocean, tsunamis normally have a very long wavelength and their amplitude is hardly noticeable. Successive waves may be from five minutes to an hour apart. They travel at speeds of around 650 km h^{-1}. However, their speed is proportional to the depth of water, which means that in coastal areas the waves slow down and increase in height, rushing onshore as highly destructive breakers. Waves have been recorded up to nearly 20 m in height above normal sea level. Large waves are most likely when tsunamis move into narrowing islets. Such waves cause terrible devastation. For example, if the wave is breaking as it crosses the shore, it can destroy houses in its path merely by the weight of the water. The subsequent backwash may carry many buildings out to sea and may remove several metres depth of sand from dune coasts.

Because of development in coastal areas within the last 50 or more years, the damaging effects of future tsunamis will probably be much more severe than in the past. Hence, it is increasingly important that the hazard is evaluated accurately and that the potential threat is estimated correctly. This involves an analysis of the risk for the purpose of planning and putting mitigation measures into place.

Nonetheless, the tsunami hazard is not frequent and, when it does affect a coastal area, its destructiveness varies both with location and time. Accordingly, an analysis of the historical record of tsunamis is required in any risk assessment. This involves a study of the seismicity of a region to establish the potential threat from earthquakes of local origin. In addition, tsunamis generated by distant earthquakes must be evaluated. The data gathered may highlight spatial differences in the distribution of the destructiveness of tsunamis that may form the basis for zonation of the hazard. If the historic record provides sufficient data, then it may be possible to establish the frequency of recurrence of tsunami events, together with the area that would be inundated by a 50, 100 or even 500 year tsunami event. On the other hand, if sufficient information is not available, then tsunami modelling may be resorted to using computer models. These provide reasonably accurate predictions of potential tsunami inundation that can be used in the management of a tsunami hazard. Such models permit the extent of damage to be estimated and the limits for evacuation to be established. The ultimate aim is to produce maps that indicate the degree of tsunami risk that, in turn, aids the planning process, thereby allowing high risk areas to be avoided or used for low intensity development. Models also facilitate the design of defense works.

Various instruments are used to detect and monitor the passage of tsunamis. These include sensitive seismographs that can record waves with long period oscillations, pressure recorders placed on the sea floor in shallow water, and buoys anchored to the sea floor and used to measure changes in the level of the sea surface. The locations of places along a coast affected by tsunamis can be hazard mapped that, for example, show the predicted heights of tsunami at a certain location for given return intervals (e.g. 25, 50 or 100 years). Homes and other buildings can be removed to higher ground and new construction prohibited in the areas of highest risk. Resettlement of coastal communities and prohibition of development in high risk areas has occurred at Hilo, Hawaii. However, the resettlement of all coastal populations away from possible danger zones is not a feasible economic proposition. Hence, there are occasions when evacuation is necessary. This depends on estimating just how destructive any tsunami will be when it arrives on a particular coast. Furthermore, evacuation requires that the warning system be effective and that there be an adequate transport system to convey the public to safe areas.

Breakwaters, coastal embankments and groves of trees tend to weaken a tsunami wave, reducing its height and the width of the inundation zone. Sea walls may offer protection against some tsunamis. Buildings that need to be located at the coast can be constructed with reinforced concrete frames and elevated on reinforced concrete piles with open spaces at ground level (e.g. for car parks). Consequently, the tsunami may flow through the ground floor without adversely affecting the building. Buildings are usually orientated at right angles to the direction of approach of the waves, that is, perpendicular to the shore. It is, however, more or less impossible to protect a coastline fully from the destructive effects of tsunamis.

Ninety per cent of destructive tsunamis occur within the Pacific Ocean, averaging more than two each year. The Pacific Tsunami Warning System, PTWS, is a communications network covering all the countries bordering the Pacific Ocean and is designed to give advance warning of dangerous tsunamis. Clearly, the PTWS cannot provide a warning of an impending tsunami to those areas that are very close to the earthquake epicentre that is responsible for the generation of the tsunami. On the other hand, waves generated off the coast of Japan will take 10 h to reach Hawaii.

Wind Action

In arid regions, because there is little vegetation and the ground surface may be dry, wind action is much more significant, and sediment yield may be high. By itself, wind can only remove uncemented rock debris but, once armed with rock particles, the wind becomes a noteworthy agent of abrasion. The most serious problem attributable to wind action is soil erosion. This makes the greatest impact in relation to agriculture. Migrating sand dunes give rise to problems. Hazards such as migrating dunes are frequently made more acute because of the intervention of man. Commonly, the problems are most severe on desert margins where rainfall is uncertain, and there may be significant human pressure on land.

Hazard maps, such as that in Figure 7.29, showing unstable sand dunes and their migration, have been produced for development purposes in arid regions (Cooke et al., 1978). Dune characteristics, notably their form and vegetation density, can be related to their mobility, and these characteristics also can be recognized on aerial photographs. High angular fresh dunes, lacking vegetation, can be regarded as potentially or actively mobile. In contrast, broad rounded low, well vegetated dunes appear to pose a low hazard risk.

The movement of wind-blown material, not always in dune form, often gives rise to problems in many arid regions as far as settlements and agricultural land are concerned. Sand and dust storms can reduce visibility, bringing traffic and airports to a halt. During severe sand storms, the visibility can be reduced to less than 10 m. In addition, dust storms may cause respiratory problems and even lead to suffocation of animals, disrupt communications and spread disease by transport of pathogens. The abrasive effect of moving sand is most notable near the ground (i.e. up to a height of 250 mm). However, over hard man-made surfaces, the abrasion height may be higher because the velocity of sand movement and saltation increase. Structures may be pitted, fluted or grooved, depending on their orientation in relation to the prevailing wind direction. An analysis of meteorological data provides an indication of the potential of wind to move sand and silt, but the actual patterns of movement are influenced by topography, mean ground wind speed and turbulence, and the availability of material.

Movement of sand can bury obstacles in its path such as roads and railways or accumulate against large structures. Such moving sand necessitates continuous and often costly maintenance activities. Indeed, areas may be abandoned as a result of sand encroachment (Fig. 8.18). Stipho (1992) pointed out that only a few centimetres of sand on a road surface can constitute a major driving hazard. Deep burial of pipelines by sand makes their inspection and maintenance difficult, and unsupported pipes on active dunes can be left high above the ground as dunes move on, causing the pipes to move and possibly fracture.

Aerial photographs and remote sensing imagery of the same area and taken at successive time intervals can be used to study dune movement, rate of land degradation and erodibility of surfaces (Jones et al., 1986). The recognition of these various features by the use of aerial photographs should be checked in the field. Field mapping involves the identification of erosional and depositional evidence of sediment movement, which can provide useful information on the direction of sand movement but less on dust transport. For instance, indirect evidence of the wind regime can be derived from trends of active dunes. As their destabilization by human activity should be avoided, it is important that dunes be identified and monitored.

Figure 8.18

Orchard partially buried by moving sand, Hebei, Gu'an County, China.

Removal of moving sand can only happen where the quantities of sand involved are small and so can only apply to small dunes. Accordingly, a means of stabilizing mobile sand must be employed. One of the best ways to bring this about is by the establishment of a vegetative cover. Natural geotextiles can be placed over sand surfaces after seeding, to protect the seeds, to help retain moisture and to provide organic matter eventually. These geotextiles can contain seeds within them. Chemical sprays have been used to stabilize loose sand surfaces. Gravel or coarse aggregate can be placed over a sand surface to prevent its deflation. A minimum particle diameter of around 20 mm is needed for the gravels to remain unaffected by strong winds. In addition, the gravel layer should be at least 50 mm in thickness. Windbreaks are frequently used to control wind erosion and, obviously, are best developed where groundwater is near enough the surface for trees and shrubs to have access to it. Fences can be used to impound or divert moving sand. Kerr and Nigra (1952) presented a review of the objectives and methods to control wind-blown sand (Table 8.3).

Glacial Hazards

Although the potential hazards of glaciers may be appreciable, their impact on man is not significant since less than 0.1% of the world's glaciers occur in inhabited areas. Nonetheless, Reynolds (1992) divided glacial hazards into two groups, that is, those that are a direct action of ice or snow, such as avalanches, and those that give rise to indirect hazards. The latter include glacier outbursts and flooding. The time involved in the different types of glacial hazards varies significantly, as can be seen from Table 8.4.

The rapid movement of masses of snow or ice down slopes as avalanches can pose a serious hazard in many mountain areas. For example, avalanches, particularly when they contain notable amounts of debris, can damage buildings and routeways, and may lead to loss of life. Two types of avalanches are recognized; firstly, the dry snow or wet snow avalanches. Slab avalanches represent the second type, and these take place when a slab of cohesive snow fails. These tend to be the more dangerous type. Avalanche location can often be predicted from historical evidence relating to previous avalanches combined with topographical data. As a consequence, hazard maps of avalanche-prone areas can be produced.

Glacier floods result from the sudden release of water that is impounded in, on, under or adjacent to a glacier. Glacier outburst and jökulhlaup both refer to a rapid discharge of water, which is under pressure, from within a glacier. Water pressure may build up within a glacier to a point where it exceeds the strength of the ice, with the result that the ice is ruptured. In this way, water drains from water-pockets so that the subglacial drainage is increased and may be released as a frontal wave many metres in height. The quantity of discharge declines

Table 8.3. Objectives and methods of aeolian sand control (after Kerr and Nigra, 1952). With the kind permission of the American Association of Petroleum Geologists

Objectives

1. The destruction or stabilization of sand accumulations in order to prevent their further migration and encroachment
2. The diversion of wind-blown sand around features requiring protection
3. The direct and permanent stoppage or impounding of sand before the location or object to be protected
4. The rendition of deliberate aid to sand movement in order to avoid deposition over a specific location, especially by augmenting the saltation coefficient through surface smoothing and obstacle removal

Methods

The above objectives are achieved by the use of one or more types of surface modification

1. Transposing. Removal of material (using anything from shovels to bucket cranes)—rarely economical or successful, and does not normally feature in long-term plans
2. Trenching. Cutting of transverse or longitudinal trenches across dunes destroys their symmetry and may lead to dune destruction. Excavation of pits in the lee of sand mounds or on the windward side of features to be protected will provide temporary loci for accumulation
3. Planting of appropriate vegetation is designed to stop or reduce sand movement, bind surface sand, and provide surface protection. Early stages of control may require planting of sand-stilling plants (e.g. *Ammophila arenaria*, beach grass), protection of surface (e.g. mulching), seeding and systematic creation of surface organic matter. Planting is permanent and attractive, but expensive to install and maintain
4. Paving is designed to increase the saltation coefficient of wind-transported material by smoothing or hard-surfacing a relatively level area, thus promoting sand migration and preventing its accumulation at undesirable sites. Often used to leeward of fencing, where wind is unladen of sediment and paving prevents its recharge. Paving may be with concrete, asphalt or wind-stable aggregates (e.g. crushed rock)
5. Panelling, in which solid barriers are erected to the windward of areas to be protected, is designed either to stop or to deflect sand movement (depending largely on the angle of the barrier to wind direction). In general, this method is inadequate, unsatisfactory and expensive, although it may be suitable for short-term emergency action
6. Fencing. The use of relatively porous barriers to stop or divert sand movement, or to destroy or stabilize dunes. Cheap, portable and expendable structures are desirable (using, for example, palm fronds or chicken wire)
7. Oiling involves the covering of aeolian material with a suitable oil product (e.g. high-gravity oil) that stabilizes the treated surface and may destroy dune forms. It is, in many deserts, a quick, cheap and effective method

Table 8.4. Types of glacial hazard

Time involved	Hazard	Description
Hours	Glacier outburst	Catastrophic discharge of water under pressure from a glacier
	Jökulhlaup	Outburst that may be associated with volcanic activity
	Débâcle	Outburst but from a proglacial lake
	Aluvión	Catastrophic flood of mud, generally transporting large boulders
Days–weeks	Flood	Areal coverage mostly by water
Months–years	Glacier surge	Rapid increase in rate of glacier flow
Years–decades	Glacier fluctuations	Variations in ice front positions due to climatic changes

after the initial surge. Water that is dammed by ice may eventually cause the ice to become detached from the rock mass on which it rests; the ice then is buoyed up, allowing the water to drain from the ice-dammed lake. Discharges of several thousand cubic metres per second have occurred from such lakes and can cause severe damage to any settlements located downstream. These outbursts often carry huge quantities of debris. Debâcles are rapid discharges of water from proglacial moraine dammed lakes, whereas aluvions involve the rapid discharge of liquid mud that may contain large boulders. Releases of large quantities of water on occasions have led to thousands of people being killed.

Glacier fluctuations in which glaciers either advance or retreat occur in response to climatic changes. The advance of a glacier into a valley can lead to the river that occupies the valley being dammed with land being inundated as a lake forms. Hence, landuse and routeways may be affected. Relatively rapid changes in the position of the snout of a glacier are referred to as surges.

Geological-Related Hazards Induced by Man

Soil Erosion

Loss of soil due to erosion by water or wind is a natural process. Soil erosion removes the topsoil, which usually contains a varying amount of the organic matter and the finer mineral fractions in soil that provide nutrient supplies for plant growth. Unfortunately, soil erosion can be accelerated by the activity of man. It is difficult, however, to separate natural from man-induced changes in erosion rates.

Soil erosion by water is most active where the rainfall finds it difficult to infiltrate into the ground so that most of it flows over the surface and, in so doing, removes soil. The intensity of rainfall is at least as important in terms of soil erosion as the total amount of rainfall and, as rainfall becomes more seasonal, the total amount of erosion tends to increase. Some sheet flow can have catastrophic effects. Such conditions are met most frequently in semi-arid and arid regions. Nevertheless, soil erosion occurs in many different climatic regions where vegetation has been removed, it being at a maximum when intense rainfall and the vegetation cover are out of phase, that is, when the surface is bare. For example, notable soil erosion can occur in humid areas that are seasonally stripped of vegetation for crop cultivation.

Sheet flow may cover up to 50% of the surface of a slope during heavy rainfall but erosion does not take place uniformly across a slope. Linear concentrations of flow may occur within sheet wash. The depth of sheet wash, up to 3 mm, and the velocity of flow are such that both laminar and turbulent flow take place. Erosion due to turbulent flow only occurs where flow is confined. The flow elsewhere is laminar and non-erosive. The velocity of sheet flow ranges between 15 to 300 mm s^{-1}. Velocities of 160 mm s^{-1} are required to erode soil particles of 0.3 mm diameter, but velocities as low as 20 mm s^{-1} will keep these particles in suspension.

Rills and gullies form when the velocity of flow increases to speeds in excess of 300 mm s^{-1}, and flow is turbulent. Whether rills or gullies form depends on soil factors, as well as velocity and depth of water flow. Rills and gullies remove much larger volumes of soil per unit area than sheet flow. Severe soil erosion, associated with the formation of gullies, can give rise to mass movements on the steepened slopes at the sides of these gullies.

Generally, an increase of erosion occurs with increasing rainfall, and erosion decreases with increasing vegetation cover. However, the growth of natural vegetation depends on rainfall, producing a rather complex variation of erosion with rainfall. Agriculture can make rainfall and vegetation partially independent of each other. Accordingly, increased erosion resulting from farming practices depends on the change in vegetation cover, the total rainfall at periods of low cover and the intensity of rainfall. Semi-arid regions show greater proportionate changes in erosion rates with rainfall than other environments and are very sensitive to small changes in climate.

The exhaustion of the organic matter in soil and soil nutrients used by plants is closely allied to soil erosion. Organic matter in the soil fulfils a similar role to clays in holding water and inorganic and organic nutrients. Organic matter is also very important in maintaining soil aggregates and in providing a moist soil that remains highly permeable. High permeability and aggregate strength minimize the risk of overland flow. Loss of organic matter depends largely on the vegetation cover and its management. Partial removal of vegetation or wholesale clearance reduces or prevents the addition of plant debris as a source of new organic material for the soil. Over a period of years, this results in a loss of plant nutrients and in a "dry" climate there is a significant reduction in soil moisture. The process can turn a semi-arid area into a desert in less than a decade. This organic depletion leads to lower infiltration capacity and increased overland flow with consequent erosion on slopes of more than a few degrees.

As far as water erosion is concerned, conservation is directed mainly at the control of overland flow, since rill and gully erosion will be reduced effectively if overland flow is prevented. There is a critical slope length at which erosion by overland flow is initiated. Hence, if slope length is reduced, for example, by terraces that follow the contours, then overland flow should be controlled. Drainage ditches must be incorporated into a terrace system to remove excess water. Contour farming involves following the contours to plant rows of crops. As with terraces, this interrupts overland flow and thereby reduces its velocity, and again can help conserve soil moisture. Gullies can be dealt with in a number of ways. Most frequently, some attempt is made at revegetating them by grassing or planting trees. Small gullies can be filled or partially filled, or traps or small dams can be constructed to collect and control movement of sediment.

Soil or crop management practices include crop rotation where a different crop is grown on the same area of land in successive years in a four or five year cycle. This avoids exhausting the soil and can improve the texture of the soil. The use of mulches, that is, covering the ground with plant residues, affords protection to the soil against raindrop impact and reduces the effectiveness of overland flow. Reafforestation of slopes, where possible, also helps to control the flow of water on slopes and reduce the impact of rainfall.

Wind erosion is most effective in arid and semi-arid regions where the ground surface is relatively dry and vegetation is absent or sparse. The problem is most acute in those regions where land-use practices are inappropriate and rainfall is unreliable. This means that the ground surface may be left exposed. Nonetheless, soil erosion by wind is not restricted to dry lands. It also occurs, though on a smaller scale, in humid areas.

Wind erosion depends on the force that the wind can exert on soil particles. In addition, the roughness of the surface over which wind blows has an important influence on erosion. For example, where the surface is rough because of large stones, the wind speed near the surface is low, and little erosion occurs. By contrast, uncultivated fields are susceptible to wind erosion, especially when the soil contains appreciable silt-size material. Losses of up to 10 mm a^{-1} were experienced in the Dustbowl of Kansas in the 1930s.

The redistribution and resorting of particles by wind erosion may have profound effects on the soils affected, their related microtopography and any agricultural activity associated with them. The process operates in a variety of natural environments that lack the protective cover of vegetation. Its human consequences are most serious in agricultural areas that experience low, variable and unpredictable rainfall; high temperatures and rates of evaporation; high wind velocity, as in semi-arid areas; as well as some of the more humid regions that experience periodic droughts. In such areas, the natural process of wind erosion may be accelerated by imprudent agricultural practices leading to soil damage and related problems.

As far as arid and many semi-arid lands are concerned, yields are generally low near desert margins, so that it is rarely worth spending large sums of money on conservation. It is in these regions that good management is necessary, and on low slopes it involves controlling yields at low sustainable levels, and varying production with wet and dry years. On steep slopes or where there are migrating dunes, vegetation must be established and grazing discouraged. To say the least, such policies are difficult to put into effect.

The problem of wind erosion can be mitigated against by reducing the velocity of the wind and/or altering the ground conditions so that particle movement becomes more difficult or particles are trapped. Windbreaks interrupt the length over which wind blows unimpeded and thereby reduce the velocity of the wind at the ground surface. The spacing, height, width, length and shape of windbreaks influence their effectiveness. Field cropping practices may be simpler and cheaper than windbreaks, and sometimes more effective. The effectiveness of the protection against wind erosion offered by plant cover is affected by the degree of ground cover provided and therefore by the height and density of the plant cover. Plants should cover over 70% of the ground surface to afford adequate protection against erosion. Vegetative cover helps trap soil particles and is particularly important in terms of reducing the amount of saltating particles. Mulches may be used to cover the soil and should be applied at a minimum rate of 0.5 kg m^{-2}. As well as providing organic matter for the soil, mulches also trap soil particles and conserve soil moisture.

The assessment of soil erosion hazard is a special form of land resource evaluation, the purpose of which is to identify areas of land where the maximum sustained productivity from a particular use of land is threatened by excessive loss of soil. Such an assessment subdivides a region into zones according to the type and degree of erosion hazard. This can be represented in map form and used as the basis for planning and conservation within a region.

The first types of soil erosion surveys consisted of mapping sheet wash, rills and gullies within an area, frequently from aerial photographs. Indices such as gully density were used to estimate erosion hazard. Subsequently, sequential surveys of the same area were undertaken in which mapping, again generally from aerial photographs, was done at regular time intervals.

Hence, changes in those factors responsible for erosion could be evaluated. They also could be evaluated in relation to other factors such as changing agricultural practices or land-use. Any data obtained from imagery should be checked and supplemented with extra data from the field.

Waste Disposal

Many types of waste material are produced by society of which domestic waste, commercial waste, industrial waste, mining waste and radioactive waste are probably the most notable. Over and above this, waste can be regarded as non-hazardous and hazardous. Deposited waste can undergo changes through chemical reactions, resulting in dangerous substances being developed, thus posing health risks and environmental problems if not managed properly. Indeed, waste disposal is one of the most expensive environmental problems to deal with. Dealing with the waste problem is one of the fundamental tasks of environmental protection.

The best method of waste disposal is determined on the basis of the type and amount of waste on the one hand, and the geological conditions of the waste disposal site on the other. In terms of locating a site, a desk study is undertaken initially. The primary task of the site exploration that follows is to determine the geological and hydrogeological conditions. Their evaluation provides the basis of models used to test the reaction of the system to engineering activities. Chemical analysis of groundwater, and at times mineralogical analysis of soils and rocks, may help yield information about their origin and hence about the future development of the site.

Although domestic waste is disposed of in a number of ways, quantitatively the most important method is placement in a sanitary landfill (Fig. 8.19). Domestic refuse is a heterogeneous collection of almost anything, much of which is capable of reacting with water to give a liquid rich in organic matter, mineral salts and bacteria. The organic carbon content is especially important since this influences the growth potential of pathogenic organisms. Leachate is formed when rainfall infiltrates a landfill and dissolves the soluble fraction of the waste, as well as from the soluble products formed as a result of the chemical and biochemical processes occurring within the decaying wastes (Loehr and Haikola, 2003). Generally, the conditions within a landfill are anaerobic, so leachates often contain high concentrations of dissolved organic substances. Barber (1982) estimated that a small landfill site with an area of 1 hectare located in southern England could produce up to 8 m^3 of leachate per day. A site with an area ten times as large would produce a volume of effluent with approximately the same biochemical oxygen demand, BOD, per year as a small rural sewage treatment plant. Hence, the location and management of these sites must be carefully controlled. Clearly, the production of leachate may represent a health hazard, for example, by polluting a groundwater

Figure 8.19

Cellular construction of a landfill for Kansas City, United States. The most common way of controlling the development of leachate is to minimize the amount of water infiltrating into a landfill, hence well-designed landfills usually have a cellular structure.

supply that would then require costly treatment prior to use (see Chapter 4). Furthermore, since it can also threaten the surface water resource potential of a region, an economic problem of considerable magnitude may be created. Consequently, a physical separation between waste on the one hand, and ground and surface water on the other, as well as an effective surface water diversion drainage system are fundamental to the design of a landfill.

Barber (1982) identified three classes of landfill sites based on hydrogeological criteria (Table 8.5). When assessing the suitability of a site, two of the principal considerations are the ease with which any pollutant can be transmitted through the substrata and the distance it is likely to spread from the site. Consequently, the primary and secondary permeabilities of the formations underlying the landfill area are of major importance. It is unlikely that the first type of site mentioned in Table 8.5 would be considered suitable. There would also be grounds for an objection to a landfill site falling within the second category of Table 8.5 if the site was located within the area of diversion to a water supply well. Generally, the third category, in which the leachate is contained within the landfill area, is to be preferred. Since all natural materials possess some degree of permeability, total containment can only be achieved if an artificial impermeable lining is provided over the bottom of the site.

Table 8.5. Classification of landfill sites based on their hydrogeology (after Barber, 1982). With kind permission of the Water Research Council

Designation	Description	Hydrogeology
Fissured site, or site with rapid subsurface liquid flow	Material with well-developed secondary permeability features	Rapid movement of leachate via fissures, joints or through coarse sediments. Possibility of little dispersion in the groundwater or attenuation of pollutants
Natural dilution, dispersion and attenuation of leachate	Permeable materials with little or no significant secondary permeability	Slow movement of leachate into the ground through an unsaturated zone. Dispersion of leachate in the groundwater; attenuation of pollutants (sorption, biodegradation, etc.) probable
Containment of leachate	Impermeable deposits such as clays or shales, or sites lined with impermeable materials or membranes	Little vertical movement of leachate. Saturated conditions exist within the base of the landfill

However, there is no guarantee that clay, soil cement, asphalt or plastic lining will permanently remain impermeable. Thus, the migration of materials from the landfill site into the substrata will occur eventually, although the length of time before this happens is subject to uncertainty. In some instances, the delay will be sufficiently long for the polluting potential of the leachate to be greatly diminished. One of the methods of tackling the problem of pollution associated with landfills is by dilution and dispersal of the leachate. Otherwise, leachate can be collected by internal drains within a landfill and conveyed away for treatment.

Selection of a landfill site for a particular waste or a mixture of wastes involves a consideration of economic and social factors as well as the hydrogeological conditions. As far as the latter are concerned, most argillaceous sedimentary, massive igneous and metamorphic rock formations have low intrinsic permeability and therefore afford the most protection to water supply. In contrast, the least protection is provided by rocks intersected by open fissures or in which solution features are developed. Coarse-grained soils may act as filters leading to dilution and decontamination. Hence, sites for disposal of domestic refuse can be chosen where decontamination has the maximum chance of reaching completion and where groundwater sources are located far enough away to enable dilution to be effective. The position of the water table is important as it determines whether wet or dry tipping takes place, as is the thickness of unsaturated material underlying a potential site. Unless waste is inert, wet tipping should be avoided. The hydraulic gradient determines the direction and velocity of the flow of leachates when they reach the water table and is also related to both the dilution that the leachates undergo, and to the points at which flow is discharged. Gray et al. (1974)

recommended that at dry sites, tipping could take place on coarse-grained material that has a thickness of 15 m or more, while any water wells should be located at least 0.8 km away.

Carbon dioxide and methane are generated by the decomposition of organic materials within sanitary landfills. However, the amount of gas produced by domestic waste varies appreciably, and a site investigation is required to determine the amount if such information is required. Nonetheless, it has been suggested that between 2.2 and 250 l kg^{-1} dry weight (0.0022 to 0.25 m^3 kg^{-1}) may be produced (Oweis and Khera, 1990). Both the gases mentioned are toxic, and methane forms a highly explosive mixture with air. Unfortunately, there are cases on record of explosions occurring in buildings due to the ignition of accumulated methane derived from the landfills on which they were built. Accordingly, planners of residential developments should avoid such sites. Proper closure of a landfill site can require gas management to control methane gas by passive venting, power-operated venting or the use of an impermeable barrier.

Protection of groundwater from the disposal of toxic waste can be brought about by containment. A number of containment systems have been developed that isolate wastes and include compacted clay barriers, slurry trench cut-off walls, geomembrane walls, diaphragm walls, sheet piling, grout curtains and hydraulic barriers (Mitchell, 1986). A compacted clay barrier consists of a trench that has been backfilled with clay compacted to give a low hydraulic conductivity. Slurry trench cut-off walls are narrow trenches filled with soil–bentonite mixtures that extend downwards into an impermeable layer (Bell et al., 1996). Again, they have a low hydraulic conductivity. In a geomembrane wall, a U-shaped geomembrane is placed in a trench and filled with sand so that its shape conforms to that of the trench. Monitoring wells are placed in the trench so that they can detect and abstract any leakage from the fill. Diaphragm walls are constructed in a similar manner to slurry trenches but are an expensive form of containment. Their use is therefore restricted to situations in which high structural stability is required. If leakage occurs via interlocks in steel sheet pile cut-offs, then their watertightness is impaired. In such instances, wells may be needed to ensure effectiveness. Grout curtains may be used in certain situations and are likely to consist of three rows of holes, the outer rows being grouted first, followed by the inner to seal any voids. Extraction wells are used to form hydraulic barriers and are located so that the contaminant plume flows towards them.

When a new site has been chosen, Mitchell (1986) maintained that a properly designed and constructed liner and cover offer long-term protection for ground and surface water. Clay liners are suitable for the containment of many wastes because of the low hydraulic conductivity of clay and its ability to absorb some wastes. They are constructed by compaction in lifts of about 150 mm thickness. Care must be taken during construction to ensure that the clay is placed at the specified moisture content and density, and to avoid cracking due to drying out during and after construction. Much care also is needed in the construction of geomembrane liners so that the seams are properly welded and the geomembrane is not torn. Composite liners incorporate both clay blankets and geomembranes. Although resistant to many chemicals, some geomembranes are susceptible to degradation by organic solvents. Hence, the United States Environmental Protection Agency recommends the use of double liner systems (Fig. 8.20). An alternative requirement recommended by Gray et al. (1974) was that a site handling toxic wastes should be underlain by at least 15 m of impermeable strata. Any well abstracting groundwater for domestic use and confined by such impermeable strata should be more than 2 km away. Yesilnacar and Cetin (2005) examined the various factors in southeastern Anatolia, Turkey, that need to be taken account of in site selection for hazardous waste disposal, including geology, topography, land-use, climate and the likelihood of earthquakes, this being vital for agricultural and water management.

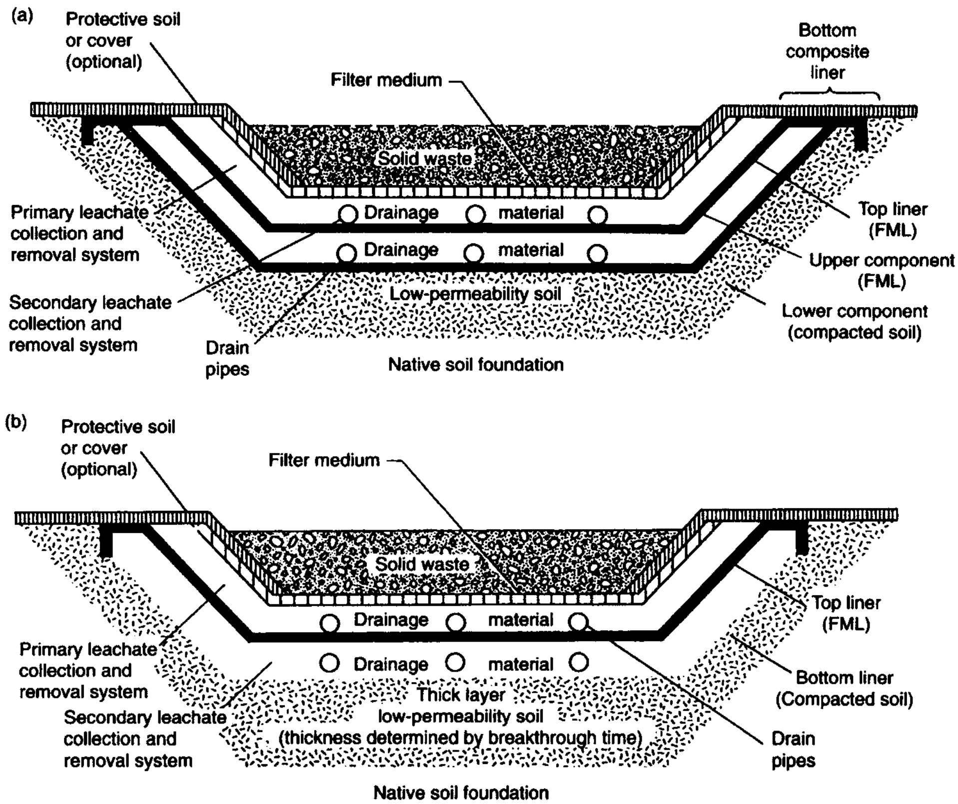


Figure 8.20

Double-liner systems proposed by the United States Environmental Protection Agency guidelines. The leachate collection layer is also considered to function as the geomembrane protection layer. (a) The flexible membrane liner, FML/composite double liner. (b) FML/compacted soil double liner (after Mitchell, 1986). With the kind permission of the Geological Society.

Disposal of liquid hazardous waste has been undertaken by injection into deep wells located in rock below freshwater aquifers, thereby ensuring that pollution of groundwater supplies does not occur. In such instances, the waste is generally injected into a permeable bed of rock several hundreds or even thousands of metres below the surface, which is confined by relatively impervious formations. However, even where geological conditions are favourable for deep well disposal, the space for waste disposal frequently is restricted and the potential injection zones are usually occupied by connate water. Accordingly, any potential formation into which waste can be injected must possess sufficient porosity, permeability, volume and confinement to guarantee safe injection. Also, the piezometric pressure in the injection zone influences the rate at which the reservoir can accept liquid waste. A further point to consider is that induced seismic activity has been associated with the disposal of fluids in deep wells (see the following text).

Monitoring is especially important in deep well disposal that involves toxic or hazardous materials. Effective monitoring requires that the geological and hydrogeological conditions be accurately evaluated and mapped before the disposal programme is started. A system of observation wells sunk into the subsurface reservoir concerned in the vicinity of the disposal well allows the movement of waste to be monitored. In addition, shallow wells sunk into fresh water aquifers permit monitoring of water quality, so that any upward migration of the waste can be noted readily.

Radioactive waste may be of low or high level. Low level waste contains small amounts of radioactivity and so does not present a significant environmental hazard if properly dealt with. Although many would not agree, it would appear that low-level radioactive waste can be disposed of safely by burying in carefully controlled and monitored sites where the hydrological and geological conditions severely limit the migration of radioactive material.

High level radioactive materials need to be separated from biological systems for hundreds or even thousands of years before they cease to represent hazards to health. Their disposal therefore presents one of the most acute problems of the present day (Eriksson, 1989). The disposal of high level liquid waste can be achieved by solidifying and mixing it with inert material. It is then placed in steel and concrete containers that may be stored in underground caverns. Permanent storage in thick impermeable rock formations such as salt, shale, granite or basalt more than 500 m below the surface is usually regarded as the most feasible means of disposal (Hunsche and Hampel, 1999). The location should be in geologically stable areas. Deep structural basins are considered as possible locations for disposal.

The necessary safety of a permanent repository for radioactive wastes has to be demonstrated by a site analysis that takes account of the site geology, the type of waste and their interrelationship. The site analysis must assess the thermomechanical load capacity of the host rock so that disposal strategies can be determined. It must determine the safe dimensions of an underground chamber and evaluate the barrier systems to be used. According to the multi-barrier concept, the geological setting for a waste repository must be able to make an appreciable contribution to the isolation of waste over a long period of time. Hence, the geological and tectonic stability (e.g. mass movement or earthquakes), the load-bearing capacity (e.g. settlement or cavern stability), geochemical and hydrogeological development (e.g. groundwater movement and potential for dissolution of rock) are important aspects of safety. Subsequently, Langer and Heusermann (2001) emphasized the importance of an analysis of the geotechnical stability and integrity of a permanent repository since they represent an important part of any safety assessment of a radioactive waste disposal project. As such, this requires the development of a geomechnical model.

As far as the disposal of high level radioactive waste in caverns is concerned, thick deposits of salt have certain advantages. Salt has a high thermal conductivity and will rapidly dissipate any heat associated with high level nuclear waste; it possesses gamma ray protection similar to concrete; it undergoes only minor changes when subjected to radioactivity; and it tends not to provide paths of escape for fluids or gases. The attractive feature of deep salt deposits is their lack of water and the inability of water from an external source to move through them. These advantages may be compromised if the salt contains numerous interbedded clay or mudstone horizons, open cavities containing brine or faults cutting the salt beds and providing conduits for external water. The solubility of salt requires that unsaturated waters be totally isolated from underground openings in beds of salt by watertight linings, isolation seals or cut-offs and/or by collection systems. If suitable precautions are not taken in more soluble horizons in salt, any dissolution that occurs can lead to the irregular development of a cavern being excavated. Any water that does accumulate in salt will be a concentrated brine that, no doubt, will be corrosive to metal canisters. Rock salt is a material that exhibits short and long-term creep. Hence, caverns in rock salt are subject to convergence as a result of plastic deformation of the salt. If creep deformation is not restrained or not compensated for by other means, excessive rock pressures can develop on a lining system, which may approach full overburden pressure.

Not all shales are suitable for the excavation of underground caverns in that soft compaction shales present difficulties in terms of wall and roof stability. Caverns may also be subject to floor heave. Caverns can be excavated in competent cemented shales. Not only do these possess low permeability, they could also adsorb ions that move through it. A possible disadvantage is that if temperatures in a cavern exceeded 100°C, clay minerals could lose water and, therefore, shrink. This could lead to the development of fractures.

Granite is less easy to excavate than rock salt or shale but is less likely to suffer problems of cavern support. It provides a more than adequate shield against radiation and will disperse any heat produced by radioactive waste. The quantity of groundwater in granite masses is small and its composition is generally non-corrosive. However, fissure and shear zones do occur within granites along which copious quantities of groundwater can flow.

The large thicknesses of lava flows in basalt plateaux mean that such successions also could be considered for disposal sites. As with granite, basalt also can act as a shield against radiation and can disperse heat. Frequently, the contact between flows is tight and little pyroclastic material is present. Joints may not be well developed at depth and, strengthwise, basalt should support a cavern. Such basalt formations can be interrupted by feeder dykes and sills that may be associated with groundwater. Furthermore, the durability of certain basalts on exposure may be suspect (Haskins and Bell, 1995).

Waste from mines is deposited on the surface as large heaps that disfigure the landscape. The configuration of a spoil heap depends on the type of equipment used in its construction and the sequence of tipping the waste. Spoil heaps associated with coal mines consist of run-of-mine material that reflects the various rock types that are extracted during mining operations. It contains varying amounts of coal that has not been separated by the preparation process. Some spoil heaps, especially those with high contents of coal, may be partially burnt or burning. The spontaneous combustion of carbonaceous material may be aggravated by the oxidation of pyrite. Spontaneous combustion can give rise to hot spots in a spoil heap where the temperatures may average around 600°C but temperatures as high as 900°C may occur. Obviously, this can cause difficulties when restoring spoil heaps. In addition, noxious gases are emitted from burning spoil, including carbon monoxide, carbon dioxide, sulphur dioxide and, less frequently, hydrogen sulphide. Each may be dangerous if breathed at certain concentrations that may be present at fires on spoil heaps. Spontaneous combustion may give rise to subsurface cavities in colliery spoil heaps. Burnt ashes may also cover zones that are red-hot to appreciable depths. When steam comes in contact with red-hot carbonaceous material, watergas is formed, and when the latter is mixed with air, over a wide range of concentrations, it becomes potentially explosive. If a cloud of coal dust is formed near burning spoil when reworking a heap, then this also can ignite and explode. Compaction, digging out, trenching, injection with non-combustible material and water, blanketing, and water spraying are methods used to control spontaneous combustion.

Obviously, an important factor in the construction of a spoil heap is its stability, which includes its long-term stability. Slips in spoil heaps represent a hazard, one of the most notable being that which occurred in 1966 at Aberfan, South Wales, resulting in 144 deaths, mainly of young children. The landslide took place on after heavy rain and, as it moved down-slope, it turned into a mudflow that flowed at approximately 32 km h^{-1}, engulfing a school and several houses (Bishop, 1973).

Old spoil heaps represent the most notable form of dereliction associated with subsurface mining. They are particularly conspicuous and can be difficult to rehabilitate into the landscape. These derelict areas need to be restored to sufficiently high standards to create an acceptable environment. The restoration of such derelict land requires a preliminary reconnaissance of the site, followed by a site investigation. The investigation provides essential input for the design of remedial measures. Restoration of a spoil heap is an exercise in large-scale earthmoving and invariably involves spreading the waste over a larger area so that gradients on the existing site can be reduced by the transfer of spoil to adjacent land (Bell, 1996b). Water courses may have to be diverted, as may services, notably roads. After an old spoil heap has been regraded, the actual surface needs restoring. Where the land is to be used for amenity or recreational purposes, the soil fertility must be restored so that the land can be grassed and trees planted. If buildings are to be erected, however, the ground must be adequately compacted, so that bearing capacities are acceptable and the buildings are not subjected to adverse settlement.

Tailings are fine-grained slurries that result from crushing rock containing metal ore, from mineral processing or are produced by the washeries at collieries (Fig. 8.21). The water in tailings may contain certain chemicals associated with the metal recovery process, such as cyanide in tailings from gold mines, and heavy metals in tailings from copper-lead-zinc mines.

Figure 8.21

Part of a tailings impoundment for spent Athabasca tar sands, Fort McMurray, Alberta, Canada.

Tailings may also contain sulphide minerals such as pyrite that can give rise to acid mine drainage. Accordingly, contaminants carried in the tailings represent a source of pollution for both ground and surface water, as well as soil.

Tailings are deposited as slurry, generally in specially constructed tailings impoundments. Tailings dams usually consist of raised embankments. The design of tailings dams obviously must pay due attention to their stability. Failure of a dam can lead to catastrophic consequences (Chandler and Tosatti, 1995). The most dangerous situation occurs when the ponded water on the discard increases in size and erodes cracks in the dam to form pipes that may emerge on the outer slopes of the dam. The failure of a dam may lead to liquefaction of the tailings (Fourie et al., 2001). What is more, these tailings are commonly contaminated. For example, the tailings dam at Aznalcollar/Los Frailes mine 45 km west of Seville, Spain, was breached in 1998, flooding approximately 4600 ha of land along the Ros Agrio and Guadiamar rivers with around 5.5 million m^3 of acidic water and 1.3×10^6 m^3 of heavy metal-bearing tailings (Hudson-Edwards et al., 2003). A catastrophic failure of a tailings dam used for the disposal of slurry from coal mining occurred at Buffalo Creek, West Virginia, in 1972. After heavy rain, water overtopped the dam and it failed, releasing 7×10^5 m^{-3} of tailings and water within a few minutes. More than 1500 houses were destroyed and 118 people killed.

The objectives of rehabilitation of tailings impoundments include their long-term mass stability, long-term stability against erosion, prevention of environmental contamination and a return of the area to productive use. Normally, when the discharge of tailings comes to an end, the level of the phreatic surface in the embankment falls as water replenishment ceases. This results in an enhancement of the stability of embankment slopes. Particular precautions need to be taken where long-term potential for environmental contamination exists. For example, as the water level in the impoundment declines, the rate of oxidation of any pyrite present in the tailings increases, reducing the pH and increasing the potential for heavy metal contamination. In the case of tailings from uranium mining, radioactive decay of radium gives rise to radon gas. Radon is a carcinogenic gas. Diffusion of radon gas does not occur in saturated tailings but, after abandonment, radon reduction measures may be necessary. In the United States, after a uranium tailings lagoon is filled it has to be covered with 3 m of earth so that the emission of radon does not exceed 2pCi m^{-2} s^{-1}.

Once the discharge of tailings ceases, the surface of the impoundment is allowed to dry. Drying of the decant pond may take place by evaporation and/or by drainage to an effluent plant. Desiccation and consolidation of the slimes may take considerable time. Stabilization can begin once the surface is firm enough to support equipment and normally involves the establishment of a vegetative cover.

Subsidence due to Mining

The subsidence effects of mineral extraction are governed by the type of deposit, the geological conditions, in particular, the nature and structure of the overlying rocks or soils, the mining methods and any mitigative action taken. In addition to subsidence due to the removal of support, mining often entails the lowering of groundwater levels that, since the vertical effective stress is raised, may cause consolidation of overburden. What is more, dewatering associated with mining in the gold-bearing reefs of the Far West Rand, South Africa, which underlie dolostone and unconsolidated deposits, produces differential subsidence over large areas and leads to the formation of sinkholes (Fig. 8.22).

Many ore deposits are concentrated in veins or ovoid bodies, the mining of which may cause localized subsidence. Transmission of a ground loss to the surface entails spreading its effect beyond the edges of the mined area and can indeed fracture the surface. Hence, the removal of a deep ore body is liable to cause subsidence over a wider area than one of equal thickness at shallow depth, although, in the former case, the lowering of the ground surface will be less severe.

Figure 8.22

Collapse of the crusher plant into a sinkhole at West Driefontein Mine, Far West Rand, South Africa, with the loss of 29 lives.

The mining method is a very important consideration, not least because of the control it exerts over the amount of mineral actually extracted during mining operations. The mining of coal or iron ore prior to the sixteenth century in Britain usually took the form of outcrop workings or bell-pitting. The latter are shaft-like excavations up to about 12 m deep that extend down into the mineral seam. Once in the seam, the workings extend outwards for a short distance. Bell-pitting leaves an area of highly disturbed ground that generally requires treatment or complete excavation before it is suitable for use for any other purpose.

Later, coal workings were usually undertaken by the pillar and stall method. This method entails leaving pillars of the mineral in place to support the roof of the workings. The method also is used to work stratiform deposits including limestone, gypsum, anhydrite, salt and sedimentary iron ores (Fig. 8.23). The amount of mineral actually mined depends primarily on maintaining the stability of the excavation so that large pillars and small stalls are formed in weaker rock masses. However, in many old coal workings, pillars were robbed prior to abandonment of the mine, increasing the possibility of pillar collapse. Slow deterioration and failure of pillars may take place years after mining operations have ceased. Old pillars at shallow depth have failed occasionally near faults, and they may fail if they are subjected to the effects of subsequent longwall mining of coal. The yielding of a large number of pillars can bring about a shallow broad trough-like subsidence over a large area (Fig. 8.24).

Figure 8.23

Exposed old pillared workings in the Bethaney Falls Limestone, near Kansas City, United States.

Figure 8.24

A trough-shaped area of subsidence due to failure of pillars at an abandoned mine, Witbank Coalfield, Mpumalanga Province, South Africa.

Even if pillars in old shallow workings are relatively stable, the surface can be affected by void migration. Void migration develops if roof rock falls into the worked-out zones and often represents the main problem in areas of shallow abandoned mines. It can occur within a few months or a very long period of years after mining has ceased. The material involved in the fall bulks, so that migration is eventually arrested, although the bulked material never completely fills the voids (Fig. 8.25). Nevertheless, the process can, at shallow depth, continue upwards to the surface, leading to the sudden appearance of a crown hole.

Assessments of mining hazards have usually been on a site basis, regional assessments being much less common. Nonetheless, regional assessments can offer planners an overview of the problems involved and can help them avoid imposing unnecessarily rigorous conditions on developers in areas where they really are not warranted. Hazard maps of areas of old mine workings frequently recognize safe and unsafe zones. The safe zones are commonly defined in terms of the thickness of cover rock, this being regarded as thick enough to preclude subsidence hazards. Unsafe zones are those in which subsidence events may occur. Development is prohibited in zones designated unsafe. Intermediate zones may be recognized between the safe and unsafe zones, where buildings are constructed with reinforced foundations or rafts, as an added precaution against unforeseen problems.

Figure 8.25

An example of void migration above a coal seam in an opencast mine, County Durham, England.

Alternatively, the permissible heights of proposed buildings in these zones are restricted in accordance with depth of mining below the site. These restrictions are progressively relaxed up to the point where they no longer apply (Stacey and Bell, 1999).

Where a site that is proposed for development is underlain by shallow old mine workings, there are a number of ways in which the problem can be dealt with. The first and most obvious method is to locate any proposed structures on sound ground away from old workings or over workings proved to be stable. Such location is not always possible. If old coal mine workings are at very shallow depth, then it might be feasible, by means of bulk excavation, to found on the strata beneath. Where the allowable bearing capacity of the foundation materials has been reduced by mining, it may be possible to use a raft. Reinforced bored pile foundations also have been resorted to in areas of abandoned mine workings. In such instances, the piles bear on a competent stratum beneath the workings. Where old mine workings are believed to pose an unacceptable hazard to development and it is not practical to use adequate measures in design or found below their level, then the ground itself can be treated. Such treatment involves filling the voids in order to prevent void migration and/or pillar collapse. For example, grouting can be undertaken by drilling holes from the surface into the mine workings and filling the remnant voids with an appropriate grout mix. Steel mesh reinforcement or geonets have been used in road construction over areas of potential mining subsidence. If a void should develop under the road, the reinforcing layer is meant to prevent the road from collapsing into the void.

The mining of coal by longwall methods is a more recent innovation that developed as a mechanized mining system during the twentieth century. It involves total extraction of a panel of coal in a seam. The working face is temporarily supported, the support being moved as the face advances, leaving the roof from which support has been withdrawn to collapse. The resulting subsidence is largely contemporaneous with mining, producing more or less direct effects at the surface (Bell, 1988a).

The surface effects of longwall mining include not only lowering but also tilting and both compressive and tensile ground strains (Fig. 8.26). As longwall mining proceeds, the ground is subject to tilting accompanied by tension and then compression. Once the subsidence front

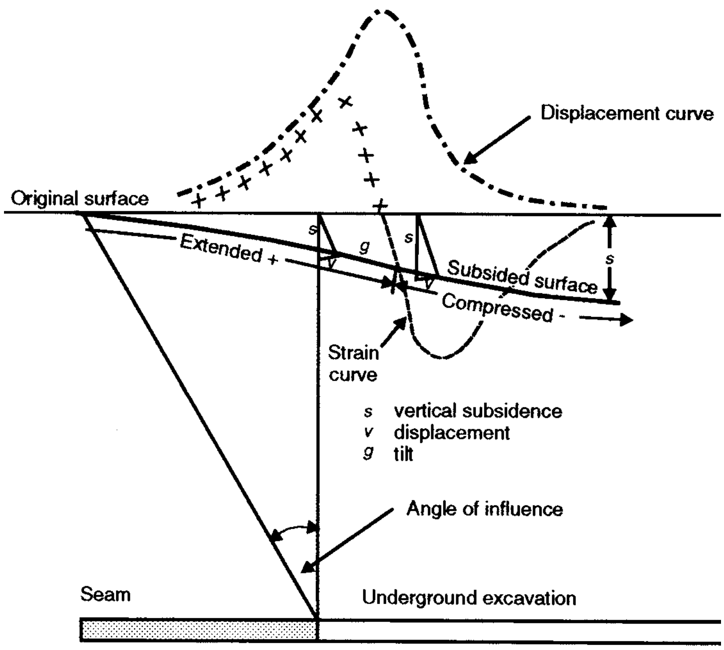


Figure 8.26

Curve of subsidence showing tensile and compressive strains, vertical subsidence and tilt, together with angle of draw (not to scale).

has passed by, the ground attains its previous slope and the ground strain returns to zero. However, permanent ground strains affect the ground above the edges of the extracted panel. For total mineral extraction and a low depth-to-width extraction ratio, the maximum amount of subsidence may be up to about 0.9 times the thickness of the seam. Normally, however, the amount of surface subsidence is significantly less than this amount.

Structural damage is not simply a function of ground strain. The shape, size and form of construction are also important factors. In many instances, subsidence effects have also been affected by the geological structure, notably the presence of faults and the character of the rocks and soils above the workings (Bell and Fox, 1988). Fault reactivation associated with mining can pose a notable problem at times. An extensive review of fault reactivation has been provided by Donnelly (2005).

The most common method of mitigating longwall subsidence damage is by the introduction of flexibility into a structure. In flexible design, structural elements deflect according to the subsidence profile (Anon, 1975b; Anon, 1977b). Flexibility can be achieved by using specially designed rafts. The use of piled foundations in areas of mining subsidence presents its own problems, in that pile caps tend to move in a spiral fashion, and each cap moves at a different rate and in a different direction according to its position relative to the mining subsidence. Hence, it is often necessary to allow the structure to move independently of the piles by the provision of a pin joint or roller bearing at the top of each pile cap. Preventative techniques can frequently be used to reduce the effects of movements on existing structures. In the case of long buildings, damage can be reduced by cutting them into smaller, structurally independent units. Pillars can be left in place to protect the surface structures above them. Maximum subsidence can also be reduced by packing or stowing the mined-out area.

Subsidence due to the Abstraction of Fluids

Subsidence due to the withdrawal of groundwater has developed with most effect in those groundwater basins where there is intensive abstraction. Such subsidence is attributed to the consolidation of sedimentary deposits in which the groundwater is present, consolidation occurring as a result of increasing effective stress (Bell, 1988b). The total overburden pressure in saturated deposits is borne by their granular structure and the pore water. When groundwater abstraction leads to a reduction in pore water pressure by draining water from the pores, this means that there is a gradual transfer of stress from the pore water to the granular structure. Put another way, the effective weight of the deposits in the dewatered zone increases since the buoyancy effect of the pore water is removed. For instance, if the water table is lowered by 1 m, it gives rise to a corresponding increase in average effective overburden pressure of 10 kPa. As a result of having to carry this increased load, the granular structure may deform in order to adjust to the new stress conditions. In particular, the porosity of the deposits concerned undergoes a reduction in volume, the surface manifestation of which is subsidence. Subsidence occurs over a longer period of time than that taken for abstraction. However, in aquifers composed of sand and/or gravel, the consolidation that takes place due to the increase in effective pressure is more or less immediate.

The amount of subsidence that occurs is governed by the increase in effective pressure, the thickness and compressibility of the deposits concerned, the length of time over which the increased loading is applied, and possibly the rate and type of stress applied. For example, the most noticeable subsidences in the Houston-Galveston region of Texas have occurred where the declines in head have been largest and where the thickness of clay in the aquifer system is greatest (Buckley et al., 2003). Furthermore, the ratio between maximum subsidence and groundwater reservoir consolidation is related to the ratio between the depth of burial and the lateral extent of the reservoir. In other words, small reservoirs that are deeply buried do not give rise to noticeable subsidence, even if subjected to considerable consolidation. By contrast, extremely large underground reservoirs may develop appreciable subsidence.

The rate at which consolidation occurs depends on the rate at which the pore water can drain from the system that, in turn, is governed by its permeability. For instance, the low permeability and high specific storage of aquitards and aquicludes under virgin stress conditions means that the escape of water and resultant adjustment of pore water pressures is slow and time-dependent. Consequently, in fine-grained beds, the increase in stress that accompanies the decline in head becomes effective only as rapidly as the pore water pressures are lowered toward equilibrium with the pressures in adjacent aquifers. The time required to reach this stage varies directly according to the specific storage and the square of the thickness of the zone from which drainage is occurring and inversely according to the vertical permeability of the aquitard. In fact, it may take months or years for fine-grained beds to adjust to increases in stress. Moreover, the rate of consolidation of slow-draining aquitards reduces with time and is usually small after a few years of loading. Maps showing the rates of subsidence, accurate to a few millimetres per year, can be drawn for periods currently up to a decade long. For example, Bell et al. (2002) reported using InSAR to investigate land subsidence in the Las Vegas area, Nevada. They were able to show that subsidence was located within four basins, each bounded by faults of Quaternary age.

In addition to being the most prominent effect in subsiding groundwater basins, surface fissuring and faulting may develop suddenly and therefore pose a greater potential threat to surface structures (Fig. 8.27). In the United States, such fissuring and faulting has occurred, especially in the San Joaquin Valley, the Houston–Galveston region and in central Arizona. These fissures, and more particularly the faults, frequently occur along the periphery of the basin. The faults are high-angled normal faults, with the downthrow on the side towards

Figure 8.27

Earth fissure in south central Arizona. The fissure results from erosional enlargement of tension cracks caused by differential subsidence. The subsidence is caused by declining groundwater levels. Courtesy of Dr T.L. Holzer.

the centre of the basin. Generally, displacements along the faults are not great, less than a metre, but movements may continue over a period of years. Holzer et al. (1979) related the annual variations in the rate of faulting to annual fluctuations in groundwater levels. Such faults do not extend beneath the zone where stresses due to lowering of the groundwater level occur.

It is not only falling groundwater levels that cause problems, a rising or high water table can be equally troublesome. Since the mid-1960s the rate of abstraction from the Chalk and the Sherwood Sandstone, the two principal aquifers in England, has decreased significantly, so that water levels are now rising (by as much as 1 m a^{-1} in places in the Chalk). The potential consequences of this includes leaks in tunnels and deep basements, reductions in the pile capacity with the possibility of structural settlement, damage to basement floors and the disruption of utility conduits. A rise in groundwater level can lead to a rise in the ground surface, for example, there has been over 20 mm of rebound since 1970 in the Venice area, although recovery of the original ground surface level is never complete. The closure of a mine with the cessation of pumping ultimately results in the flooding of the old workings and subsequent rebound of groundwater levels until equilibrium is reached (Younger, 2002). Groundwater rebound may rise and discharge at the surface from fault outcrops, mine entries, or fractured or permeable strata.

Surface subsidence also occurs in areas where there is intensive abstraction of oil, as well as natural gas. Again subsidence is attributed to the consolidation of the fluid-bearing formations that result from the increase in vertical effective stress. In most cases, and particularly in clayey deposits, the subsidence does not occur simultaneously with the abstraction of fluid. More than 40 known examples of differential subsidence, horizontal displacement or surface faulting have been associated with 27 oil and gas fields in California and Texas. The most spectacular case of subsidence is that of the Wilmington Oilfield near Long Beach, California. By 1966, after 30 years of production, an elliptical area of more than 75 km^2 had subsided more than 8.8 m. The Groningen Gasfield in the Netherlands has been in production since 1965. Regular precision levelling surveys have been carried out since abstraction of gas commenced in order to monitor the amount of subsidence occurring at ground surface. Contour maps showing the amount of subsidence have been produced from this data. Another method of subsidence monitoring has involved measuring the consolidation of near-surface sediments by using borehole extensometers.

Deposits that readily go into solution, particularly salt, can be extracted by solution mining. For example, salt has been obtained by brine pumping in a number of areas in the United Kingdom, Cheshire being by far the most important (Bell, 1992c). Subsidence due to salt extraction by wild brine pumping still continues to be an inhibiting factor, on a very much reduced scale, as far as developments in Cheshire are concerned. Wild brine pumping was carried out on the major natural brine runs and active subsidence was normally concentrated at the head and sides of a brine run where freshwater first entered the system. Hence, serious subsidence occurred at considerable distances, up to 8 km from pumping centres, and was of unpredictable nature. In addition, tension cracks and small fault scars formed in the surface tills on the convex flanks of subsidence hollows (Fig. 8.28). Because the exact area from which salt was extracted was not known, the magnitude of subsidence developed could not be related to the volume of salt worked. Consequently, there was no accurate means of predicting the amount of ground movement or strain. Sulphur is mined in Texas and Louisiana by the Frasch process that involves pumping hot water into the beds of sulphur and then pumping brine to the surface. Subsidence troughs with associated small faults at the periphery of the basins have formed.

Poorly drained areas, especially low-lying marshes and swamplands, generally contain high proportions of organic matter in their associated soils. When such soils are drained,

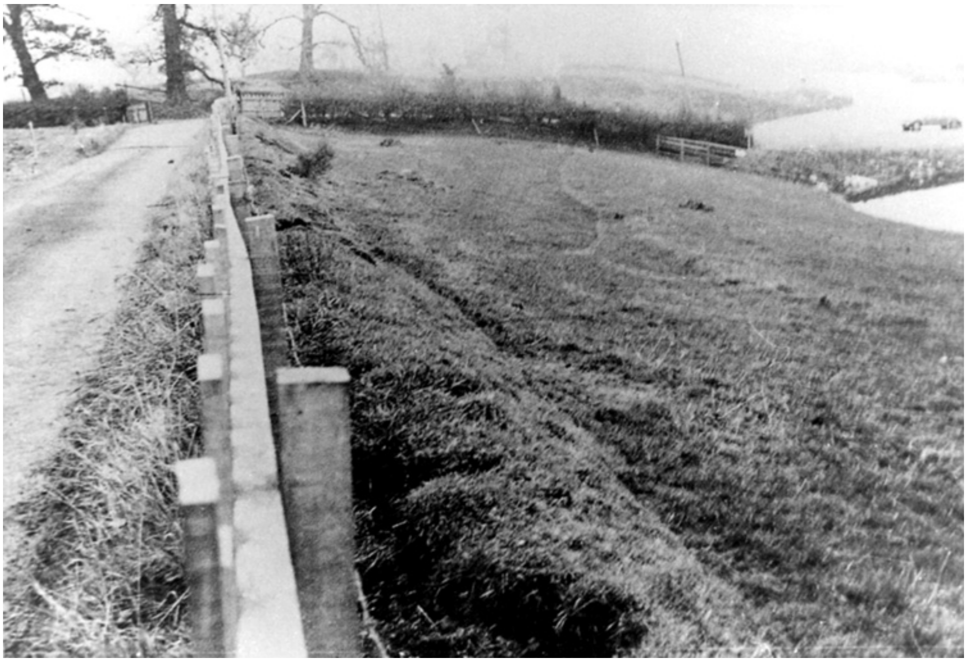


Figure 8.28

A tension scar produced by wild-brine pumping, Cheshire, England. Note the flash (lake) in the top right-hand corner.

the ground level subsides. The subsidence is not only due to the consolidation that occurs as a result of the loss of the buoyant force of groundwater but is also attributable to desiccation and shrinkage associated with drying out in the zone of aeration and oxidation of the organic material in that zone. Peat deposits have by far the greatest potential for subsidence when drained (Fig. 8.29). In fact, because peat is highly porous, with moisture contents that can range up to 2000%, it is by far the most compressible soil type.

Induced Sinkholes

Many sinkholes are induced by man's activities, that is, they result from declines in groundwater level, especially those due to excessive abstraction. For example, Jammal (1986) recorded that 70 sinkholes appeared in Orange and Seminole counties in central Florida over a period of 20 years. Most of these developed in those months of the year when rainfall was least (i.e. April and May) and withdrawal of groundwater was high. Nonetheless, the appearance of a sinkhole at the surface represents a late-stage expression of processes that have been in operation probably for thousands of years.

Figure 8.29

Subsidence caused by drainage of peat, Benwick, Cambridgeshire, England. Note the house on the right where the ground floor window is at pavement level, the house next door has been demolished and the others are tilted at various angles.

Most collapses forming sinkholes result from roof failures of cavities in unconsolidated deposits above karstic limestone (Waltham and Fookes, 2003). These cavities are created when the unconsolidated deposits move or are eroded downwards into openings in the top of bedrock. The stability of residual soils that overlie cavernous limestone is a concern during the location and construction of buildings (Drumm and Yang, 2005). Collapse of bedrock roofs, as compared with the migration of unconsolidated deposits into openings in the top of bedrock, is rare.

Some induced sinkholes develop within hours of the effects of man's activity being imposed on the geological and hydrogeological conditions. Several collapse mechanisms have been proposed and have included the loss of buoyant support to roofs of cavities or caverns in bedrock previously filled with water; increase in the amplitude of water-level fluctuations; increases in the velocity of movement of groundwater; and movement of water from the land surface to openings in underlying bedrock where most recharge had previously been rejected since the openings were occupied by water.

Dewatering associated with mining in the gold-bearing reefs of the Far West Rand, South Africa, which underlie dolostone and unconsolidated deposits, has led to the formation of sinkholes and produced differential subsidence over large areas (Fig. 8.22). Hence, certain areas became unsafe for occupation. It then became a matter of urgency that the areas that were subject to the occurrence of sinkholes be delineated. Sinkholes formed concurrently with the lowering of the water table in areas that formerly, in general, had been free from sinkholes (De Bruyn and Bell, 2001). A sinkhole develops when the self-supporting arch of unconsolidated material collapses into a cavity in the dolostone beneath. Initially, the neck of the cavity was small enough to permit the unconsolidated deposits to bridge across it. Normally, the cover of unconsolidated material is less than 15 m, otherwise it will choke the cavity on collapse.

Areas underlain by highly cavernous limestones possess most sinkholes, hence sinkhole density has proven a useful indicator of potential subsidence. As there is preferential development of solution voids along zones of high secondary permeability because these concentrate groundwater flow, data on fracture orientation and density, fracture intersection density and the total length of fractures have been used to model the presence of solution cavities in limestone. Therefore, the locating of areas of high risk of cavity collapse has been done by using the intersection of lineaments formed by fracture traces and lineated depressions. Aerial photographs have proved particularly useful in this context. Brook and Alison (1986) described the production of subsidence susceptibility maps of a covered karst terrain in Dougherty County, southwest Georgia. These maps were developed using a geographical information system that incorporated much of the type of data referred to. The microgravity technique has been used by Styles et al. (2005) to characterize cavities in karst terrain. Buttrick et al. (2001) developed a method for assessing the risk of sinkhole hazard in dolostone areas in South Africa.

Induced Seismicity

Induced seismicity occurs where changes in the local stress conditions brought about by man give rise to changes in strain and deformation in rock masses, causing movements along discontinuities. These movements may be on a microscopic or macroscopic scale. Most events of this type have been associated with large reservoirs, underground liquid waste disposal, hydrocarbon extraction, underground mining or large explosions. Such activities through the single or combined effect of unloading, loading or increased pore pressures in the rock masses concerned allow the release of stored strain energy. The earthquakes produced tend to have magnitudes of less than 5, although more serious events have occurred. For instance, an earthquake of magnitude 6.5 was associated with the Koyna Reservoir in India and resulted in loss of life and damage to the dam itself. The area in which Koyna Reservoir is located in characterized by wrench faulting. Evidence suggests that it was already in a critical state of stress before the dam was constructed, and impounding of the reservoir provided the necessary triggering mechanism for movements to occur along the faults due to the increase in pore water pressure. Induced seismicity in the Denver area of Colorado was attributed to the disposal of liquid waste down a deep well. Over 700 minor earthquakes of magnitudes up to 4.3 were recorded. Water injection to enhance the recovery of oil has also led to the occurrence of seismic events.

Seismic emissions may be brought about as a result of the fluid acting as a lubricant in microcracks and fissures on the one hand, and by increasing the pore pressures, thereby reducing the shear strength of the rock mass on the other. Both may trigger the release of strain energy from the rocks and facilitate movements along faults. It has been suggested that there is a critical pore fluid pressure required for earthquake activity to rise above background levels.

Violent rock failures (i.e. rock bursts) are associated with deep mining. The latter causes stress changes that lead to the sudden release of stored strain energy. In the gold mines of the Witwatersrand system in South Africa, such release of strain energy is responsible for the generation of seismic events that range up to magnitudes of around 4 (De Bruyn and Bell, 1997). The sources of most of the seismic events are located in the rock mass in the immediate vicinity of mining activity. Minor seismicity has also been caused by coal mining. In the latter part of the 1970s, minor earth tremors were recorded in the neighbourhood of Stoke-on-Trent, England, and were linked with mining at Hem Heath Colliery (Kusznir et al., 1980). The presence of old workings in the same area appeared to be an additional causative factor.

Derelict and Contaminated Land

Derelict land can be regarded as land that has been damaged by industrial use or other means of exploitation to the extent that it has to undergo some form of remedial treatment before it can be of beneficial use. Such land usually has been abandoned in an unsightly condition and is normally located in urban areas. Not only is derelict land a wasted resource, but it also has a blighting effect on the surrounding area and can deter new development. Its rehabilitation is therefore highly desirable, not only by improving the appearance of an area but also by making a significant contribution to its economy by bringing derelict land back into worthwhile use. Accordingly, there is both economic and environmental advantage in the regeneration of derelict land. The use to which derelict land is put should suit the needs of the surrounding area and be compatible with other forms of land use.

Any project involving the rehabilitation of derelict land requires a feasibility study. This needs to consider topography and geological conditions, site history, and the local environment and existing infrastructure. The results of the feasibility study allow an initial assessment to be made of the possible ways to develop a site. This is followed by a site investigation.

Figure 8.30

A survey of derelict land being undertaken in Leeds, England.

The investigation provides essential input data for the design of remedial measures and indicates whether or not demolition debris can be used in the rehabilitation scheme.

Derelict land may present hazards, for example, disposal of industrial wastes may have contaminated land, in some cases so badly that earth has to be removed, or the ground may be severely disturbed by the presence of massive old foundations and subsurface structures such as tanks, pits and conduits for services. Contaminated land may emit gases or may represent a fire hazard. Details relating to such hazards should be determined during the site investigation. Site hazards result in constraints on the freedom of action, necessitate stringent safety requirements, may involve time-consuming and costly working procedures and affect the type of development. Derelict sites may require varying amounts of filling, levelling and regrading. As far as fill is concerned, this should be obtained from on site if possible, otherwise from an area nearby. Once regrading has been completed, the actual surface needs restoring. This is not so important if the area is to be built over (e.g. if it is to be used for an industrial estate), as it is if it is to be used for amenity or recreational purposes. In the case in which buildings are to be erected, however, the ground must be adequately compacted so that they are not subjected to adverse settlement. On the other hand, where the land is to be used for amenity or recreational purposes, then soil fertility must be restored so that the land can be grassed and trees planted.

Some of the worst dereliction has been associated with past mineral workings and mining activities. The presence of old shafts represents a particular problem (Anon, 1982; Bell, 1988c). Abandoned quarries, pits and spoil heaps are particularly conspicuous, and can be difficult to rehabilitate into the landscape. Quarries are often screened from view by planting trees around them or, where conditions allow, used for waste disposal. Some large, old quarry voids have been used for storage or for the location of industrial units. In limestone areas in particular, landform replication represents the best method of quarry reclamation. Landform replication involves the construction of landforms and associated habitats similar to those of the surrounding environment (Bailey and Gunn, 1991). Restoration blasting along quarry faces can be used to replicate scree slopes and to produce multi-faceted slope sequences. At times, old mineral workings can be turned into industrial museums, used for storage or become tourist attractions (Fig. 8.31).

The extraction of sand and gravel deposits in low-lying areas along the flanks of rivers frequently means that the workings eventually extend beneath the water table. On rehabilitation, it is not necessary to fill the flooded pits completely. Partial filling and landscaping can convert such sites into recreational areas offering such facilities as sailing, fishing and other water sports. It is necessary to carry out a thorough survey of flooded workings, with soundings being taken so that accurate plans and sections can be prepared. The resultant report then forms the basis for the design of the measures involved in rehabilitation (Bell and Genske, 2000).

Opencast working of coal involves excavation to depths of up to around 100 m below the surface. Working is advanced on a broad front with face lengths of 3 to 5 km not being uncommon. The material excavated, minus the coal, is used to fill the void. It is placed above ground in a suitable position in relation to void filling so that later rehandling is minimized. Restoration can begin before a site is closed, indeed this usually is the more convenient method. Hence, worked out areas behind the excavation front are filled with rock waste. This means that the final contours can be designed with less spoil movement than if the two operations were undertaken separately (Hughes and Clarke, 2003). The restored land is generally used for agriculture or forestry but it can be used for country parks, golf courses, etc. The water table at many opencast sites is lowered by pumping in order to provide dry working conditions in the pit. If a site is to be restored for agricultural use or for forestry, then this takes place without compaction control. However, if a site is to be built over, then settlement is likely to be a problem without proper compaction. Significant settlements of opencast backfill can occur when the partially saturated material becomes saturated by rising groundwater after pumping has ceased (Charles et al., 1993).

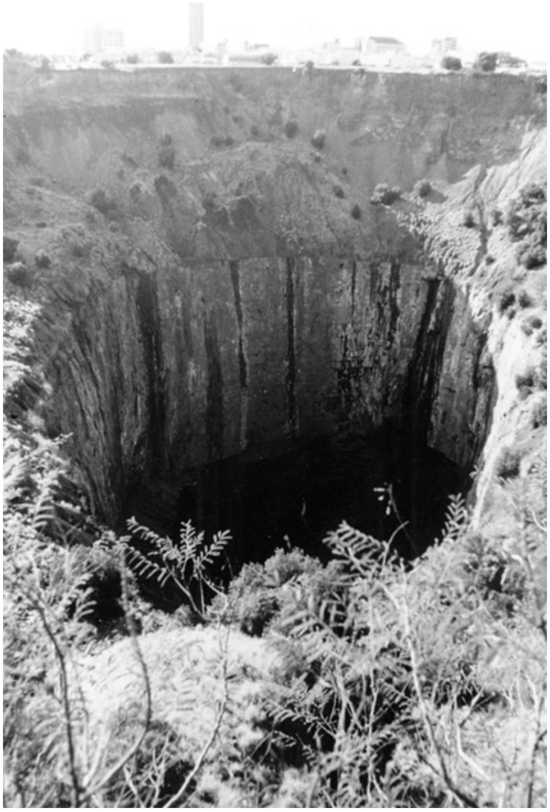


Figure 8.31

The Big Hole, Kimberley, South Africa. This famed old diamond working is now a popular tourist attraction.

Land can contain substances that are undesirable or even hazardous as a result of natural processes, for example, as a result of mineralization. However, most cases of contaminated land are associated with human activity. In particular, in many of the industrialized countries of the world, one of the legacies of the past two centuries is that land has been contaminated. Hence, when such sites are cleared for redevelopment, they can pose problems. Contaminated land is by no means easy to define but it can be regarded as land that contains substances that, when present in sufficient concentrations, are likely to cause harm, directly or indirectly, to man, to the environment or to other targets.

Contamination can take many forms and can be variable in nature across a site, and each site has its own characteristics (Attewell, 1993). In some cases, only a single previous use of a site may be identified, which has a characteristic pattern of contamination. On the other hand, some sites have had a number of former uses, especially when industry was established on them over a long period of time. In such cases, there may be no particular characteristic pattern of contamination present. The types of contaminants that may be encountered include heavy metals, sulphates, asbestos, various organic compounds, toxic and flammable gases, combustible materials and radioactive materials.

The migration of soil-borne contaminants is associated primarily with groundwater movement, and the effectiveness of groundwater to transport contaminants is dependent mainly on their solubility. The quality of water can provide an indication of the mobility of contaminants and the rate of dispersal. Liquid and gas contaminants, of course, may be mobile.

The first stage in any investigation of a site suspected of being contaminated is a desk study that provides data for the design of the subsequent ground investigation. The desk study should identify past and present uses of the site, and the surrounding area, and the potential for and likely forms of contamination. The objectives of the desk study are to identify any hazards and the primary targets likely to be at risk; to provide data for health and safety precautions for the site exploration; and to identify any other factors that may act as constraints on development. Hence, the desk study should provide, wherever possible, information on the layout of the site, including structures below ground; its physical features; the geology and hydrogeology of the site; the previous history of the site; the nature and quantities of materials likely to be handled; the processing involved; health and safety records; and methods of waste disposal. It should allow a preliminary risk assessment to be made. Just as a normal site investigation, one that is involved with the exploration for contamination needs to determine the nature of the ground. In addition, it needs to assess the ability of the ground to transmit any contaminants either laterally, or upward by capillary action. Permeability testing is therefore required. The exploration must also establish the location of any perched water tables and aquifers, and any linkages between them, as well as determine the chemistry of the water on site. The exploratory methods used in the site exploration can include manual excavation, trenching and the use of trial pits, light cable percussion boring, power auger drilling, rotary drilling, and water and gas surveys (Bell et al., 1996). Sampling procedures are of particular importance. Digitization of data for the production of various site plans can be accomplished by a geographical information system.

A wide range of technologies are available for the remediation of contaminated sites, and the applicability of a particular method depends on the site conditions, the type and extent of contamination, and the extent of remediation required. In some situations, it may be possible to rely on natural decay or dispersion of the contaminants. Removal of contaminants from a site for disposal in an approved disposal facility has frequently been used. However, the costs involved in removal can be extremely high. On-site burial can be carried out by the provision of an acceptable surface barrier or an approved depth of clean surface filling over the contaminated material. This may require considerable earthwork. Clean covers are most appropriate for sites with various previous uses and contamination. They should not be used to contain oily or gaseous contamination. Another method is to isolate the contaminants by in situ containment by using, for example, cut-off barriers. Encapsulation involves immobilization of contaminants, for instance, by the injection of grout. Low grade contaminated materials can be diluted below threshold levels by mixing with clean soil. However, there are possible problems associated with dilution. The need for quality assurance is high (unacceptable materials cannot be reincorporated into the site). Other less used methods include bioremediation of organic material, soil flushing and soil washing, incineration, vacuum extraction and venting of volatile constituents or removal of harmful substances to be buried in containers at a purpose-built facility.

Geology and Construction

Geology is one of the most important factors in construction since construction takes place either at or below the ground surface. Hence, geology has an influence on most construction operations because it helps determine their nature, form and cost.

Open Excavation

Open excavation refers to the removal of material, within certain specified limits, for construction purposes. In order to accomplish this economically and without hazard, the character of the rocks and soils involved and their geological setting must be investigated. Indeed, the method of excavation and the rate of progress are influenced very much by the geology on site (Kentli and Topal, 2004). Furthermore, the stability of the sides of an excavation and the position of the water table in relation to the base level of an excavation are of importance, as are any possible effects of construction operations on the surrounding ground and/or buildings (Finno et al., 2005).

A Note on Slope Stability

The stability of slopes is a critical factor in open excavation. This is particularly the case in cuttings, as for instance, for roads, canals and railways, where slopes should be designed to resist disturbing forces over long periods. In other words, a stability analysis should determine under what conditions a proposed slope will remain stable. For a further note on slope stability, see Chapter 3.

Instability in a soil mass occurs when slip surfaces develop and movements are initiated within it. Undesirable properties in a soil, such as low shearing strength, development of fissures and high pore water pressure, tend to encourage instability and are likely to lead to deterioration in slopes. In the case of open excavation, removal of material can give rise to the dissipation of residual stress that can aid instability.

There are several methods available for analysis of the stability of slopes in soils (Morgenstern, 1995). Most of these may be classed as limit equilibrium methods, in which the basic assumption is that the failure criterion is satisfied along the assumed path of failure. Starting from known or assumed values of the forces acting upon the soil mass, calculation is made of the shear resistance required for equilibrium of the soil. This shear resistance then is compared with the estimated or available shear strength of the soil to give an indication of the factor of safety, assessed in two dimensions instead of all three. The analysis gives a conservative result.

The design of a slope excavated in a rock mass requires as much information as possible on the character of the discontinuities within the rock mass, since its stability is frequently dependent on the nature of the discontinuities (Hoek and Bray, 1981). Information relating to the spatial relationships between discontinuities affords some indication of the modes of failure that may occur. Information relating to the shear strength of the rock mass or, more particularly, the shear strength along discontinuities, is required for use in a stability analysis (Bye and Bell, 2001). The inclination of discontinuities is always the most important parameter for slopes of medium and large height.

Excavations in Rocks and Soils

Slopes of excavations in fresh massive plutonic igneous rocks such as granite and gabbro can be left more of less vertical after removal of loose fragments. On the other hand, volcanic rocks such as basalt and andesite generally are bedded and jointed, and may contain layers of ash, which usually are softer and weather more rapidly. Thus, slope angles have to be reduced accordingly.

Gneiss, quartzite and hornfels are highly weather resistant and slopes in them may be left almost vertical. Schist varies in character, and some of the softer types may be weathered and tend to slide along their planes of schistosity. Slate generally resists weathering, although slips may occur where the cleavage daylights into a cut face.

If strata are horizontal, then excavation is relatively straightforward, and slopes can be determined with some degree of certainty. Vertical slopes can be excavated in massive limestone and sandstone that are horizontally bedded. In brittle, cemented shale, slopes of 60–75° usually are safe, but increasing fissility and decreasing strength necessitate flatter slopes. Even in weak shale, slopes are seldom flatter than 45°. However, excavated slopes may have to be modified in accordance with the dip and strike directions in inclined strata. The most stable excavation in dipping strata is one in which the face is orientated normal to the strike, since in such situations there is a low tendency for rocks to slide along their bedding planes. Conversely, if the strike is parallel to the face, then the strata dip into one slope. This is most critical where the rocks dip at angles varying between 30 and 70°. If the dip exceeds 70°

and there is no alternative to working against the dip, then the face should be developed parallel to the bedding planes for safety reasons.

Inclined sedimentary sequences in which thin layers of shale or clay occur between beds of sandstone or limestone may have to be treated with caution, especially if the bedding planes are dipping at a critical angle. Weathering may reduce such material to an unstable state within a short period of time that, in turn, can lead to slope failure.

A slope of 1:1.5 is generally used when excavating dry sand, this more or less corresponding to the angle of repose, that is, 30–40°. This means that a cutting in a coarse soil will be stable, irrespective of its height, as long as the slope is equal to the lower limit of the angle of internal friction, provided that the slope is drained suitably. In other words, the factor of safety, *F*, with respect to sliding may be obtained from:

$$F = \frac{\tan \phi}{\tan \beta} \tag{9.1}$$

where ϕ is the angle of internal friction and β is the slope angle.

Slope failure in coarse soils is a surface phenomenon that is caused by the particles rolling over each other down the slope. As far as sands are concerned, their packing density is important. For example, densely packed sands that are very slightly cemented may have excavated faces with high angles that are stable. The water content is of paramount importance in loosely packed sands, for if these are saturated they are likely to flow on excavation.

The most frequently used gradients in many clay soils vary between 30 and 45°. In some clays, however, in order to achieve stability, the slope angle may have to be less than 20°. The stability of slopes in clay depends not only on its strength and the angle of the slope but also on the depth to which the excavation is taken and on the depth of a firm stratum, if one exists, not far below the base level of the excavation. Slope failure in a uniform clay soil takes place along a near-circular surface of slippage. For example, the critical height, *H*, to which a face of an open excavation in normally consolidated clay can stand vertically without support can be obtained from:

$$H = \frac{4c}{9.8\gamma} \tag{9.2}$$

where c is the cohesion of the clay and γ its unit weight.

In stiff fissured clays, the fissures appreciably reduce the strength to below that of intact material (Skempton, 1964). Thus, reliable estimation of slope stability in stiff fissured clays is difficult.

Generally, steep slopes can be excavated in such clays initially but their excavation means that fissures open due to the relief of residual stress, and there is a change from negative to positive pore water pressure along the fissures, the former having tended to hold the fissures together. This change can occur within a matter of days or hours. Not only does this weaken the clay but it also permits a more significant ingress of water, which means that the clay is softened. Irregular-shaped blocks may begin to fall from the face, and slippage may occur along well-defined fissure surfaces that are by no means circular. If there are no risks to property above the crests of slopes in stiff fissured clays, then they can be excavated at about 35°. Although this will not prevent slips, those that occur are likely to be small.

The stability of the floor of large excavations may be influenced by ground heave. The amount of heave and the rate at which it occurs depends on the degree of reduction in vertical stress during construction operations, on the type and succession of underlying strata and on the surface and groundwater conditions. Heave generally is greater in the centre of a level excavation in relatively homogeneous ground as, for example, clays and shales. Long-term swelling involves absorption of water from the ground surface or is due to water migrating from below. Where the excavation is in overconsolidated clays or shales, swelling and softening is quite rapid. In the case of clays with low degrees of saturation, swelling and softening take place very rapidly if surface water gains access to the excavation area.

Methods of Excavation: Drilling and Blasting

The method of excavation is determined largely by the geology of the site, however, consideration also must be given to the surroundings. For instance, drilling and blasting, although generally the most effective and economical method of excavating hard rock, are not desirable in built-up areas since damage to property or inconvenience may be caused.

The rock properties that influence drillability include hardness, abrasiveness, grain size and discontinuities. The harder the rock, the stronger the bit that is required for drilling since higher pressures need to be exerted. Abrasiveness may be regarded as the ability of a rock to wear away drill bits. This property is closely related to hardness and in addition is influenced by particle shape and texture. The size of the fragments produced during drilling operations influence abrasiveness. For example, large fragments may cause scratching but comparatively little wear, whereas the production of dust in stronger but less abrasive rock causes polishing. This may lead to the development of high skin hardness on tungsten carbide bits that, in turn, may cause them to spall. Even diamond-studded bits lose their cutting ability upon polishing. Generally, coarse-grained rocks can be drilled more quickly than fine-grained varieties or those in which the grain size is variable.

The ease of drilling in rocks in which there are many discontinuities is influenced by their orientation in relation to the drillhole. Drilling over an open discontinuity means that part of the energy controlling drill penetration is lost. Where a drillhole crosses discontinuities at a low angle, this may cause the bit to stick. It also may lead to excessive wear and to the hole going off line. Drilling across the dip is generally less difficult than drilling with it. If the ground is badly broken, then the drillhole may require casing. Where discontinuities are filled with clay, this may penetrate the flush holes in the bit, causing it to bind or deviate from alignment.

Spacing of the blastholes is determined on the one hand in relation to the strength, density and fracture pattern within the rock, and on the other in relation to the size of the charge. Careful trials are the only certain method of determining the correct burden and blasting pattern in any rock. As a rule, spacing varies between 0.75 and 1.25 times the burden. Generally, 1 kg of high explosive will bring down about 8–12 tonnes of rock. Good fragmentation reduces or eliminates the amount of secondary blasting while minimizing wear and tear on loading machinery.

Rocks characterized by high specific gravity and high intergranular cohesion with no preferred orientation of mineral grains cause difficulties in blasting. They have high tensile strength and very low brittleness values, the high tensile strength resisting crack initiation and propagation upon blasting. Examples are provided by gabbros, breccias and greenstones. A second group that provides difficulties includes those rocks, such as certain granites, gneisses and marbles, which are relatively brittle with a low resistance to dynamic stresses. Blasting in such rocks gives rise to extensive pulverization immediately about the charged holes, leaving the area between almost unfractured. These rocks do not give an effective energy transfer from the detonated charge to the rock mass. The third category of rocks giving rise to difficult blasting is those possessing marked preferred orientation, mica schist being a typical example. The difficulty arises from the influence of the mechanical anistropy due to the preferred orientation of the flaky minerals. These rocks split easily along the lineation but crack propagation across it is limited.

In many excavations, it is important to keep overbreak to a minimum. Apart from the cost of its replacement with concrete, damage to the rock forming the walls or floor may lower its strength and necessitate further excavation. What is more, smooth faces allow excavation closer to the payline and are more stable. There are two basic methods that can be used for this purpose, namely, line drilling and presplitting.

Line drilling is the method most commonly used to improve the peripheral shaping of excavations. It consists of drilling alternate holes between the main blastholes forming the edge of the excavation. The quantity of explosive placed in each line hole is significantly smaller and indeed if these holes are closely spaced, from 150 to 250 mm, then explosive may be placed only in every second or third hole. The closeness of the holes is based on the type of rock being excavated and on the payline. These holes are timed to fire ahead, with or after the nearest normally charged holes of the blasting pattern. The time of firing similarly is dependent largely on the character of the rock involved.

Pre-splitting can be defined as the establishment of a free surface or shear plane in a rock mass by the controlled usage of explosives in appropriately aligned and spaced drillholes. A line of trimming holes is charged and fired to produce a shear plane. This acts as a limiting plane for the blast proper and is carried out prior to the drilling and blasting of the main round inside the proposed break lines. The spacing of the trimming holes is governed by the type of rock and the diameter of the hole. Once pre-split, the rock excavation can be blasted with a normal pattern of holes. In most rocks, a shear plane can be induced to the bottom of the line holes, that is, to base level, but in very tight unfissured rocks, difficulty may be experienced in breaking out the main blast to base level.

Possible damage to property due to blasting vibrations can be estimated in terms of ground velocity, but it is extremely difficult to determine the limit values of ground velocity for varying degrees of damage. However, a conservative limit of 50 mm s^{-1} seems to be commonly accepted as the limit below which no damage will be caused to internal renderings and plasterwork. Nevertheless, low vibration levels may disturb sensitive machinery.

Vibrographs can be placed in locations considered susceptible to blast damage in order to monitor ground velocity. A record of the blasting effects compared with the size of the charge and distance from the point of detonation normally is sufficient to reduce the possibility of damage to a minimum (Fig. 9.1). The use of multiple-row blasting with short-delay ignition reduces the effects of vibration.

Methods of Excavation: Ripping

The major objective of ripping in construction practice is to break the rock just enough to enable economic loading to take place (Fig. 9.2). Rippability depends on intact strength, fracture index and abrasiveness, that is, strong, massive and abrasive rocks do not lend themselves to ripping (MacGregor et al., 1994). On the other hand, if sedimentary rocks such as sandstone and limestone are well bedded and jointed, or if strong and weak rocks are thinly interbedded, then they can be excavated by ripping rather than by blasting. Indeed, some of the weaker sedimentary rocks (less than 1 MPa point load strength, 15 MPa compressive strength) such as mudstones are not as easily removed by blasting as their low strength would suggest since they are pulverized in the immediate vicinity of the hole. What is

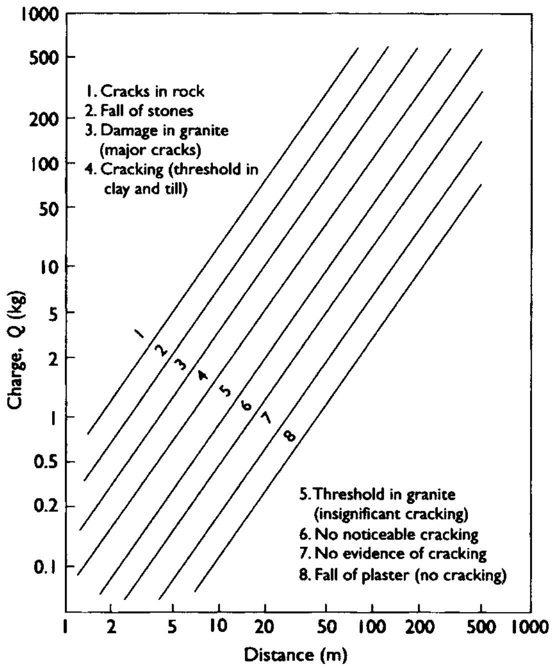


Figure 9.1

Charge *Q* as a function of distance for various charge levels.

Figure 9.2

A bulldozer with tyne attachment being used for ripping in a limestone quarry near Kansas City, United States.

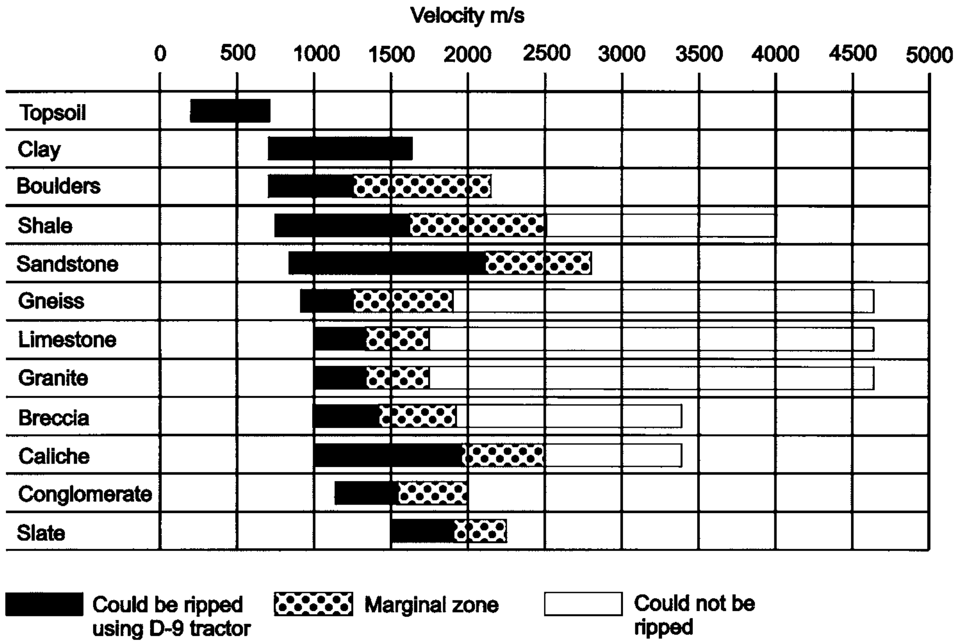


Figure 9.3

Rippability chart.

more, when blasted, mudstones may lift along bedding planes to fall back when the pressure has been dissipated. Such rocks, particularly if well jointed, are more suited to ripping.

The most common method for determining rippability is by seismic refraction. The seismic velocity of the rock mass concerned then can be compared with a chart of ripper performance based on ripping operations in a wide variety of rocks (Fig. 9.3). Kirsten (1988), however, argued that seismic velocity could only provide a provisional indication of the way in which rock masses could be excavated. Previously, Weaver (1975) had proposed the use of a modified form of the geomechanics classification as a rating system for the assessment of rock mass rippability (Table 9.1).

The run direction during ripping should be normal to any vertical joint planes, down-dip to any inclined strata and, on sloping ground, downhill. Ripping runs of 70–90 m usually give the best results. Where possible, the ripping depth should be adjusted so that a forward speed of 3 km h^{-1} can be maintained, since this generally is found to be the most productive. Adequate breakage depends on the spacing between ripper runs that are, in turn, governed by the fracture pattern in the rock mass.

Table 9.1. Rippability rating chart (after Weaver, 1975). With the permission of the Institution of Civil Engineers of South Africa

	Rock class				
	I	II	III	IV	V
Description	**Very good rock**	**Good rock**	**Fair rock**	**Poor rock**	**Very poor rock**
Seismic velocity (m/s)	> 2150	2150–1850	1850–1500	1500–1200	1200–450
Rating	26	24	20	12	5
Rock hardness (MPa)	Extremely hard rock (> 70)	Very hard rock (20–70)	Hard rock (10–20)	Soft rock (3–10)	Very soft rock (1.7–3.0)
Rating	10	5	2	1	0
Rock weathering	Unweathered	Slightly weathered	Weathered	Highly weathered	Completely weathered
Rating	9	7	5	3	1
Joint spacing (mm)	> 3000	3000–1000	1000–300	300–50	< 50
Rating	30	25	20	10	5
Joint capacity	Non-continuous	Slightly continuous	Continuous – no gouge	Continuous – some gouge	Continuous – with gouge
Rating	5	6	3	0	0
Joint gouge	No separation	Slight separation	Separation < 1 mm	Gouge < 5 mm	Gouge > 5 mm
Rating	5	5	4	3	1
Strike and dip orientation*	Very unfavourable	Unfavourable	Slightly unfavourable	Favourable	Very favourable
Rating	15	13	10	5	3
Total rating	100–90	90–70†	70–50	50–25	< 25
Rippability assessment	Blasting	Extremely hard ripping and blasting	Very hard ripping	Hard ripping	Easy ripping
Tractor selection	—	DD9G/D9G	D9/D8	D8/D7	D7
Horsepower	—	770/385	385/270	270–180	180
Kilowatts	—	575–290	290/200	200–135	135

* Original strike and dip orientation now revised for rippability assessment.
† Ratings in excess of 75 should be regarded as unrippable without preblasting.

Methods of Excavation: Digging

The diggability of ground is of major importance in the selection of excavating equipment and depends principally upon the intact strength of the ground, its bulk density, bulking factor and natural water content. The latter influences the adhesion or stickiness of soils, especially clay soils.

At present, there is no generally acceptable quantitative measure of diggability, assessment usually being made according to the experience of the operators. However, a fairly reliable indication can be obtained from similar excavations in the same materials in the area or the behaviour of the ground excavated in trial pits. Attempts have been made to evaluate the performance of excavating equipment in terms of seismic velocity (Fig. 9.4). It would appear that most earth-moving equipment operates best when the seismic velocity of the ground is less than 1000 m s^{-1} and will not function above approximately 1800 m s^{-1}.

When material is excavated, it increases in bulk, this being brought about by the decrease that occurs in density per unit volume. Some examples of typical bulking in soils are given in Table 9.2. The bulking factor is important in relation to loading and removal of material from the working face.

Groundwater and Excavation

Groundwater frequently represents one of the most difficult problems during excavation, and its removal can prove costly. Not only does water make working conditions difficult, but piping, uplift pressures and flow of water into an excavation can lead to erosion and failure of the sides.

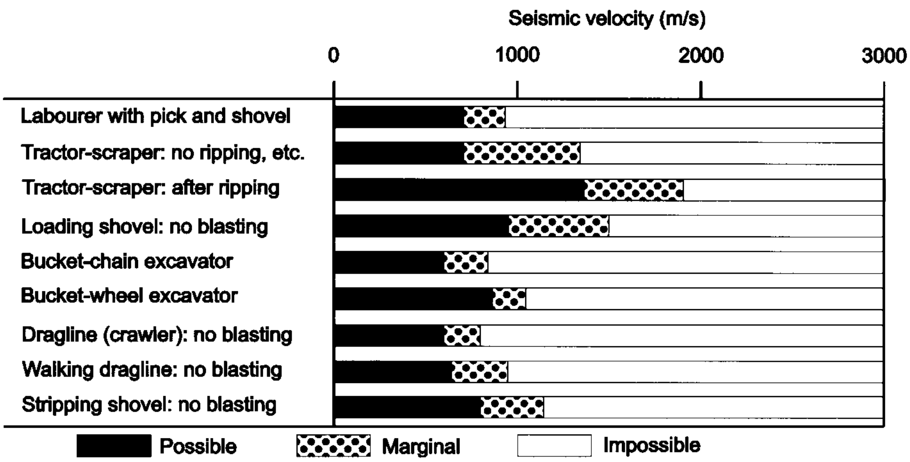


Figure 9.4

Seismic velocities for determining diggability.

Table 9.2. Density, bulking factor and diggability of some common soils

Soil type	Density (Mg m^{-3})	Bulking factor	Diggability
Gravel, dry	1.8	1.25	E
Sand, dry	1.7	1.15	E
Sand and gravel, dry	1.95	1.15	E
Clay, light	1.65	1.3	M
Clay, heavy	2.1	1.35	M–H
Clay, gravel and sand, dry	1.6	1.3	M

Note: E = easy digging, loose, free-running material such as sand and small gravel; M = medium digging, partially consolidated materials such as clayey gravel and clay; M–H = medium hard digging, materials such as heavy wet clay and large boulders.

Collapsed material has to be removed and the damage made good. Subsurface water normally is under pressure, which increases with increasing depth below the water table. Under high pressure gradients, weakly cemented rock can disintegrate. High piezometric pressures may cause the floor of an excavation to heave or, worse still, cause a blow-out (see Chapter 4). Hence, data relating to the groundwater conditions should be obtained prior to the commencement of operations.

Some of the worst conditions are met in excavations that have to be taken below the water table (Forth, 2004). In such cases, the water level must be lowered by some method of dewatering. The method adopted depends on the permeability of the ground and its variation within the stratal sequence, the depth of base level below the water table and the piezometric conditions in underlying horizons. Pumping from sumps within an excavation, bored wells or wellpoints are the dewatering methods most frequently used (Bell and Cashman, 1986). Impermeable barriers such as steel sheet piles, secant piles, diaphragm walls, frozen walls and grouted walls can be used to keep water out of excavations (Bell and Mitchell, 1986). Ideally, these structures should be keyed into an impermeable horizon beneath the excavation.

Methods of Slope Control and Stabilization

It rarely is economical to design a rock slope so that no subsequent rock falls occur, indeed many roads in rough terrain could not be constructed with the finance available without accepting some such risk. Therefore, except where absolute security is essential, slopes should be designed to allow small falls of rock under controlled conditions.

Fences supported by rigid posts can contain small rockfalls, but larger heavy duty catch fences are required for larger rockfalls. Rock traps in the form of a ditch and/or barrier can be

Figure 9.5

Wire netting fixed to a steep slope excavated in gneiss, northeast of Bergen, Norway.

installed at the foot of a slope. Benches on a slope also may act as traps to retain rock fall, especially if a barrier is placed at their edge. Wire mesh fixed to the face provides yet another method for controlling rockfall (Fig. 9.5). Where a road or railway passes along the foot of a steep slope, protection from rockfall is afforded by the construction of a rigid canopy from the face of the slope.

Excavation involving the removal of material from the head of an unstable slope, flattening of the slope, benching of the slope or complete removal of the unstable material helps stabilize a slope. If some form of reinforcement is required to provide support for a rock slope, then it is advisable to install it as quickly as possible after excavation. Dentition refers to masonry or concrete infill placed in fissures or cavities in a rock slope (Fig. 9.6). Thin-to-medium-bedded rocks dipping parallel to the slope can be held in place by steel dowels grouted into drilled holes, which are up to 2 m in length. Rock bolts may be up to 8 m in length with tensile working loads of up to 100 kN (Fig. 9.6). They are put in tension so that the compression induced in the rock mass improves shearing resistance on potential failure planes. Light steel sections or steel mesh may be used between bolts to support the rock face. Rock anchors are used for major stabilization works, especially in conjunction with retaining structures. They may exceed 30 m in length. In general, for excavated slopes it is more advantageous to improve

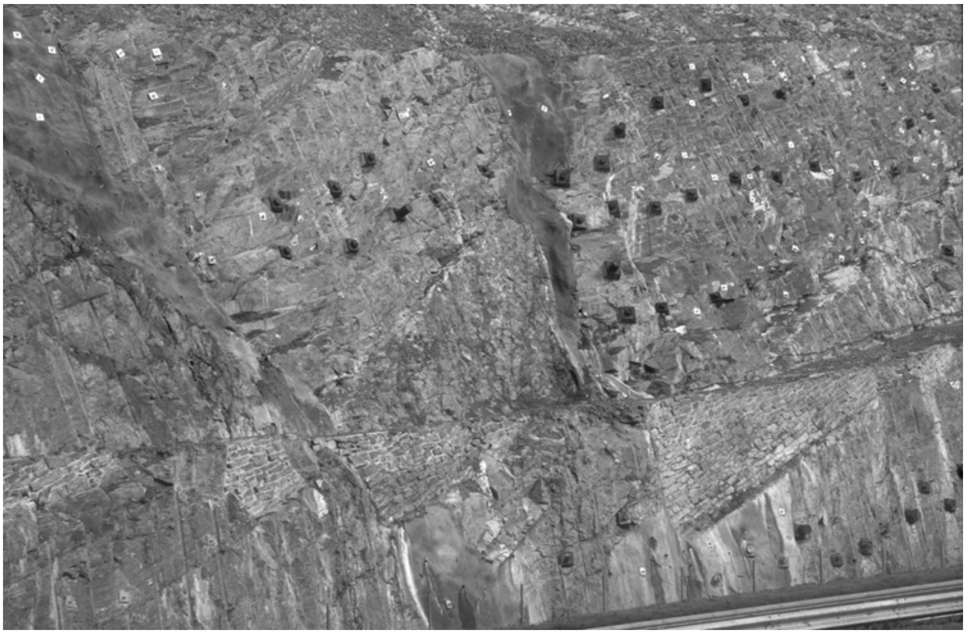


Figure 9.6

Dentition and rock bolts used to stabilize an excavation in limestone along the A55 road in North Wales.

the properties of the rock slope itself (by anchoring or bolting) than to remove the rock and replace it with concrete.

Gunite or shotcrete frequently is used to preserve the integrity of a rock face by sealing the surface and inhibiting the action of weathering (Fig. 9.7). They are pneumatically applied mortar or concrete, respectively. Coatings may be reinforced with wire mesh and used in combination with rock bolts. Heavily fractured rocks may be grouted in order to stabilize them.

Restraining structures control sliding by increasing the resistance to movement. They include retaining walls, cribs, gabions and buttresses. There are certain limitations that must be considered before retaining walls are used for slope control. These involve the ability of the structure to resist shearing action, overturning and sliding on or below the base of the structure. Retaining walls often are used where there is a lack of space for the full development of a slope, such as along many roads and railways. As retaining walls are subjected to unfavourable loading, a large wall width is necessary to increase slope stability. Reinforced earth can be used for retaining earth slopes. Such a structure is flexible and so can accommodate some settlement. Thus, reinforced earth can be used on poor ground where conventional alternatives would require expensive foundations. Reinforced earth walls are constructed by erecting a thin front skin at the face of the wall at the same time as the earth

Figure 9.7

An excavation in gneiss that has been shotcreted, Goteborg, Sweden.

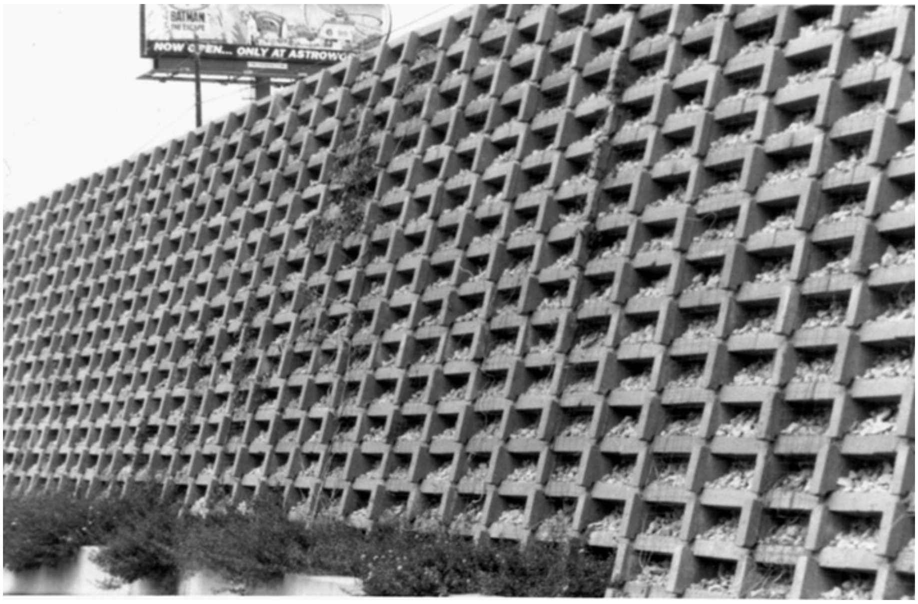
is placed (Fig. 9.8). Strips of steel or geogrid are fixed to the facing skin at regular intervals. Cribs may be constructed of precast reinforced concrete or steel units set up in cells that are filled with gravel or stone (Fig. 9.9a). Gabions consist of strong wire mesh surrounding placed stones (Fig. 9.9b). Concrete buttresses occasionally have been used to support large blocks of rock, usually where they overhang.

Geosynthetic materials, especially geomats and geogrids, are being used increasingly to protect slopes. They are draped over slopes requiring protection and are pegged onto the soil. Geomats are three-dimensional geosynthetics that, if filled with soil and seed, help to establish a vegetative cover.

Drainage generally is the most applicable method for improving the stability of slopes or for the corrective treatment of slides, regardless of type, since it reduces the effectiveness of one of the principal causes of instability, namely, excess pore water pressure. The most likely zone of failure must be determined so that the extent of the slope mass that requires drainage treatment can be defined.

Figure 9.8

Reinforced earth being used along the A82 road in Scotland.



(a)

Figure 9.9

(a) A crib wall being used to retain a slope in San Antonio, Texas.

Continued

(b)


Figure 9.9—cont'd

(b) Gabions used to retain a slope just outside Port St. Johns, South Africa.

Surface run-off should not be allowed to flow unrestrained over a slope. This usually is prevented by the installation of a drainage ditch at the top of an excavated slope to collect drainage from above. The ditch, especially if in soils, should be lined to prevent erosion, otherwise its enlargement will mean that it will act as a tension crack. It may be filled with cobble aggregate. Herringbone ditch drainage usually is employed to convey water from the surfaces of slopes. These drainage ditches lead into an interceptor drain at the foot of the slope (Fig. 9.10a). Infiltration can be lowered by sealing the cracks in a slope by regrading or filling with cement, bitumen or clay. A surface covering has a similar purpose and function. For example, the slope may be covered with granular material resting upon filter fabric.

Support and drainage may be afforded by counterfort-drains, where an excavation is made in sidelong ground, likely to undergo shallow, parallel slides. Deep trenches are cut into the slope, lined with filter fabrics and filled with granular material. The granular fill in each trench acts as a supporting buttress or counterfort, as well as providing drainage. However, counterfort drains must extend beneath the potential failure zone, otherwise they merely add unwelcome weight to the sloping mass.

Successful use of subsurface drainage depends on tapping the source of water, locating the presence of permeable material that aids free drainage, the location of the drain on relatively

(a)

(b)

Figure 9.10

(a) Surface drainage of a slope in deposits of till, near Loch Lomond, Scotland. The aggregate-filled ditch drainage leads to an interceptor drain. (b) Internal drainage gallery in restored slope, near Aberfan, South Wales.

unyielding material to ensure continuous operation (flexible PVC drains are now frequently used) and the installation of a filter to minimize silting in the drainage channel. Drainage galleries are costly to construct and in slipping areas may experience caving (Fig. 9.10b). They should be backfilled with stone to ensure their drainage capacity if partially deformed by subsequent movements. Galleries are indispensable in the case of large slipped masses where drainage has to be carried out over lengths of 200 m or more. Drillholes may be made about the perimeter of a gallery to enhance drainage. Drainage holes with perforated pipes are much cheaper than galleries and are satisfactory over short lengths, but it is more difficult to intercept water-bearing layers with them. When individual benches are drained by horizontal holes, the latter should lead into a properly graded interceptor trench, which is lined with impermeable material.

Deep wells may be used to drain slopes. Usually, the water collected is conveyed away at the base of the well but at times pumps may be installed at the bottom of a well to remove the water.

Tunnels and Tunnelling

Geology is the most important factor that determines the nature, form and cost of a tunnel. For example, the route, design and construction of a tunnel are largely dependent on geological considerations. Estimating the cost of tunnel construction, particularly in areas of geological complexity, is uncertain.

Prior to tunnel construction, the subsurface geology is explored by means of pits, adits (drifts), drilling and pilot tunnels. Exploration adits driven before tunnelling proper commences are not usually resorted to unless a particular section appears to be especially dangerous or a great deal of uncertainty exists. Core drilling aids the interpretation of geological features already identified at the surface.

A pilot tunnel is probably the best method of exploring tunnel locations and should be used if a major-sized tunnel is to be constructed in ground that is known to have critical geological conditions. It also drains the rock ahead of the main excavation. If the inflow of water is excessive, the rock can be grouted from the pilot tunnel before the main excavation reaches the water-bearing zone.

Reliable information relating to the ground conditions ahead of the advancing face obviously is desirable during tunnel construction. This can be achieved with a varying degree of success by drilling long horizontal holes between shafts, or by direct drilling from the tunnel face at regular intervals. In extremely poor ground conditions, tunnelling progresses behind an array of probe holes that fan outwards some 10–30 m ahead of the tunnel face. Although this slows progress, it ensures completion. Holes drilled upwards from the crown of the tunnel and forwards from the side walls help locate any abnormal features such as faults, buried channels, weak seams or solution cavities. Equipment for drilling in a forward direction can be incorporated into a shield or tunnel boring machine. The penetration rate of a probe drill must exceed that of the tunnel boring machine, ideally it should be about three times faster. Maintaining the position of the hole, however, presents the major problem when horizontal drilling is undertaken. In particular, variations in hardness of the ground oblique to the direction of drilling can cause radical deviations. Even in uniform ground, rods go off line. The inclination of a hole therefore must be surveyed.

Geophysical investigations can give valuable assistance in determination of subsurface conditions, especially in areas in which the solid geology is poorly exposed. Seismic refraction has been used in measuring depths of overburden in the portal areas of tunnels, in locating faults, weathered zones or buried channels, and in estimating rock quality. Seismic testing also can be used to investigate the topography of a river bed and the interface between the alluvium and bedrock when tunnels are excavated beneath rivers. Seismic logging of boreholes can, under favourable circumstances, provide data relating to the engineering properties of rock. Resistivity techniques have proved useful in locating water tables and buried faults, particularly those that are saturated. Resistivity logs of drillholes are used in lateral correlation of layered materials of different resistivities and in the detection of permeable rocks. Ground probing radar offers the possibility of exploring large volumes of rock for anomalies in a short time and at low cost, in advance of major subsurface excavations.

Geological Conditions and Tunnelling

Large planar surfaces form most of the roof in a formation that is not inclined at a high angle and strikes more or less parallel to the axis of a tunnel. In tunnels in which jointed strata dip into the side at 30° or more, the up-dip side may be unstable. Joints that are parallel to the axis of a tunnel and that dip at more than 45° may prove especially treacherous, leading to slabbing of the walls and fallouts from the roof. The effect of joint orientation in relation to the axis of a tunnel is given in Table 9.3.

The presence of flat-lying joints may also lead to blocks becoming dislodged from the roof. When the tunnel alignment is normal to the strike of jointed rocks and the dips are less than 15°, large blocks are again likely to fall from the roof. The sides, however, tend to be reasonably stable. When a tunnel is driven perpendicular to the strike in steeply dipping or vertical strata, each stratum acts as a beam with a span equal to the width of the cross section. However, in such a situation, blasting operations are generally less efficient. If the

Table 9.3. The effect of joint strike and dip orientations in tunnelling

Strike perpendicular to tunnel axis				Strike parallel with tunnel axis	
Drive with dip		Drive against dip			
Dip 45–90°	Dip 20–45°	Dip 45–90°	Dip 20–45°	Dip 45–90°	Dip 20–45°
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair

Dip 0–20° unfavourable irrespective of strike

axis of a tunnel runs parallel to the strike of vertically dipping rocks, then the mass of rock above the roof is held by the friction along the bedding planes. In such a situation, the upper boundary of loosened rock, according to Terzaghi (1946), does not extend beyond a distance of 0.25 times the tunnel width above the crown.

When the joint spacing in horizontally layered rocks is greater than the width of a tunnel, then the beds bridge the tunnel as a solid slab and are only subject to bending under their own weight. Thus, if the bending forces are less than the tensile strength of the rock, then the roof need not be supported. In conventional tunnelling, in which horizontally lying rocks are thickly bedded and contain few joints, the roof of the tunnel is flat. Conversely, if the rocks are thinly bedded and are intersected by many joints, a peaked roof is formed. Nonetheless, breakage rarely, if ever, continues beyond a vertical distance equal to half the width of the tunnel above the top of a semicircular payline (Fig. 9.11). This type of stratification is more dangerous where the beds dip at 5–10°, since this may lead to the roof spalling, as the tunnel is driven forward.

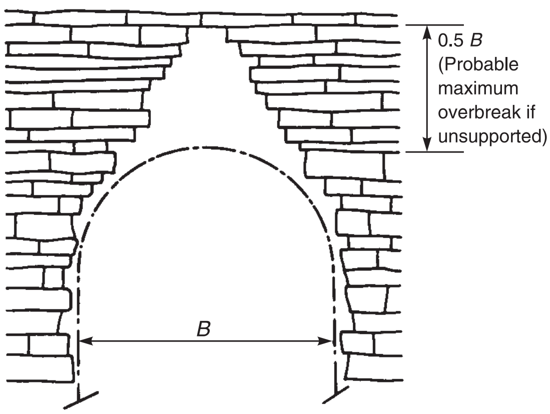


Figure 9.11

Overbreak in thinly bedded horizontal strata with joints. Ultimate overbreak occurs if no support is installed.

Weathering of rocks leads to the strength of the material being reduced, at times dramatically. However, weathering processes are rarely sufficiently uniform to give gradual and predictable changes in the engineering properties of a weathered profile. In fact, such profiles usually consist of heterogenous materials at various stages of decomposition and/or disintegration. Mudrocks are more susceptible to weathering and breakdown than most other rock types (Bell et al., 1997). The breakdown of mudrocks begins on exposure, which leads to the opening and development of fissures as residual stress is dissipated, and to an increase in moisture content and softening. Olivier (1979) developed the geodurability classification (see Chapter 3) primarily to assess the durability of mudrocks and poorly cemented sandstones during tunnelling operations, since the tendency of such rocks to disintegrate governs the stand-up time of tunnels. Some basalts and dolerites also are susceptible to rapid weathering. Bell and Haskins (1997) noted that degradation of the basalts on exposure in the Transfer Tunnel from Katse Dam, Lesotho, initially took the form of crazing, that is, extensive microfracturing. These microfractures expand with time, causing the basalt to disintegrate into gravel-sided fragments. Some minerals are prone to rapid breakdown on exposure, and their reaction products can give rise to further problems. For example, sulphur compounds, notably pyrite, on breakdown, give rise to ferrous sulphate and sulphuric acid, which are injurious to concrete. Gypsum, especially when in particulate form, can be dissolved rapidly and similarly generates sulphates and sulphuric acid. Hydration of anhydrite to gypsum produces an increase in volume. For example, Yuzer (1982) recorded swelling pressures up to 12 MPa when tunnelling through an evaporitic formation in Turkey. It is believed that no great length of time is required to bring about such hydration.

Problems of tunnel excavation and tunnel stability in areas of karstic limestone may be associated with weathered rock, abrupt changes in lithology, rock weakened by dissolution or containing discontinuites opened by dissolution, as well as with the presence of voids and caverns with or without infill. Furthermore, a large cavern in the path of a tunnel presents a difficult problem and may delay excavation or, in extreme cases, may mean that the tunnel has to be diverted around it or relocated. Milanovic (2003) gave examples of several tunnels in China in which the planned route of the tunnel had to be relocated because of the presence of large caverns through which underground streams flowed. In addition, construction of a tunnel may alter the groundwater regime of the locality, as a tunnel usually acts as a drain. This can lead to dewatering, leading to sinkhole development/reactivation, which may be manifested in the form of surface subsidence, as occurred at the Canyon Tunnel, Sri Lanka (James, 1993). Isolated heavy flows of groundwater may occur in association with solution pipes and cavities or fault zones, especially those in which the limestone has been crushed and subjected to dissolution. When water is under appreciable pressure it may break into the tunnel as a gusher. For example, Calembert (1975) reported that 1000 l s^{-1} entered the Talave Tunnel in Spain and that the flow was diminished only after several months. The water table in karstic carbonate rocks can rise rapidly after periods of sustained or heavy rainfall, sometimes by tens of metres in a number of hours. If the water table rises above the axis of a tunnel in karstic limestone with well-developed conduits, then this can lead to rapid flooding of the tunnel. Also, when tunnelling in karst terrains, primarily above the water table, sediment that has collected in cavities, voids and enlarged discontinuities may be washed into the tunnel as mud after severe storms. Again quoting Calembert (1975), more than 30 000 m^3 of debris entered the Gran Sasso Tunnel, Spain. The debris was associated with a wide faulted zone that had been subjected to dissolution. This interrupted construction for many months.

Faults generally mean non-uniform rock pressures on a tunnel and hence necessitate special treatment at times, such as the construction of box sections with invert arches. Generally, problems increase as the strike of a fault becomes more parallel to the tunnel opening. However, even if the strike is across the tunnel, faults with low dips can represent a hazard. If the tunnel is driven from the hanging wall, the fault first appears at the invert, and it generally is possible to provide adequate support or reinforcement when driving through the rest of the zone. Conversely, when a tunnel is driven from the foot-wall side, the fault first appears in the crown, and there is a possibility that a wedge-shaped block, formed by the fault and the tunnel, will fall from the roof without warning.

Major faults usually are associated with a number of minor faults, and the dislocation zone may occur over many metres. What is more, rock material within a faulted zone may be shattered and unstable. Problems tend to increase with increasing width of the fault zone. Sometimes, a fault zone is filled with sand-sized crushed rock that has a tendency to flow into the tunnel. If, in addition, the tunnel is located beneath the water table, a sandy suspension may rush into the tunnel. When a fault zone is occupied by clay gouge and a section of a tunnel follows the gouge zone, swelling of this material may occur and cause displacement or breakage of tunnel supports during construction. Large quantities of water in a permeable rock mass are impounded by a fault zone occupied by impervious gouge and are released when tunnelling operations penetrate through the fault zone.

Movements along major active faults in certain parts of the world can disrupt a tunnel lining and even lead to a tunnel being offset. As a consequence, it is best to shift the alignment to avoid the fault, or, if possible, to use open cut within the active fault.

The earthquake risk to an underground structure is influenced by the material in which it occurs. For instance, a tunnel at shallow depth in alluvial deposits will be seriously affected by a notable earthquake because of the large relative displacements of the ground surrounding it. On the other hand, a deep tunnel in solid rock will be subjected to displacements that are considerably less than those that occur at the surface. The main causes of stresses in shallow underground structures arise from the interaction between the structure and displacement of the ground. If the structure is sufficiently flexible, it will follow the displacements and deformations to which the ground is subjected.

Rocks, especially those at depth, are affected by the weight of overburden, and the stresses so developed cause the rocks to be strained. In certain areas, particularly orogenic belts, the state of stress is also influenced by tectonic factors. However, because the rocks at depth are confined, they suffer partial strain. The stress that does not give rise to strain, in other words, stress which is not dissipated, remains in the rocks as residual stress. While the rocks remain in a confined condition, the stresses accumulate and may reach high values, sometimes in excess of yield point. If the confining condition is removed, as in tunnelling, then the residual stress can cause displacement. The amount of movement depends on the magnitude of the residual stress. The pressure relief, which represents a decrease in residual stress, may be instantaneous or slow in character, and is accompanied by movement of the rock mass with variable degrees of violence.

In tunnels, driven at great depths below the surface, rock may suddenly break from the sides of the excavation. This phenomenon is referred to as rock bursting. In such failures, hundreds of tonnes of rock may be released with explosive force. Rock bursts are due to the dissipation of residual stresses that exceed the strength of the ground around the excavation, and their frequency and severity tend to increase with depth. Indeed, most rock bursts occur at depths in excess of 600 m. The stronger the rock, the more likely it is to burst. The most explosive failures occur in rocks that have unconfined compressive strengths and values of Young's modulus greater than 140 MPa and 34.5 GPa, respectively.

Popping is a similar but less violent form of failure. In this case, the sides of an excavation bulge before exfoliating. Spalling tends to occur in jointed or cleaved rocks. To a certain extent, such a rock mass can bulge as a sheet, collapse occurring when a key block either fails or is detached from the mass.

In fissile rock such as shale, the beds may slowly bend into the tunnel. In this case, the rock is not necessarily detached from the main mass, but the deformation may cause fissures and hollows in the rock surrounding the tunnel.

Another pressure relief phenomenon is bumping ground. Bumps are sudden and somewhat violent earth tremors that, at times, dislodge rock from the sides of a tunnel. They probably are due to rock displacements consequent upon the newly created stress conditions.

Tunnelling in Soft Ground

All soft ground moves in the course of tunnelling operations (Peck, 1969). In addition, some strata change their characteristics on exposure to air. Both factors put a premium on speed of advance, and successful tunnelling requires matching the work methods to the stand-up time of the ground.

As far as soft-ground tunnelling is concerned, the difficulties and costs of construction depend almost exclusively on the stand-up time of the ground and this, in turn, is influenced by the position of the water table in relation to the tunnel (Hansmire, 1981). Above the water table, the stand-up time principally depends on the shearing and tensile strength of the ground, whereas below it, it also is influenced by the permeability of the material involved. Occasionally, tunnels in soft ground may suffer partial collapse, for example, the Hull waste-water flow transfer tunnel, England, was constructed mainly in glacial soils but collapse occurred in 1999 in a relatively short stretch of alluvial soils (Grose and Benton, 2005).

Terzaghi (1950a) distinguished the following types of soft ground:

1. *Firm ground*. Firm ground has sufficient shearing and tensile strength to allow the tunnel heading to be advanced without support, typical representatives being stiff clays with low plasticity and loess above the water table.
2. *Ravelling ground*. In ravelling ground, blocks fall from the roof and sides of the tunnel some time after the ground has been exposed. The strength of the ground usually decreases with increasing duration of load. It also may decrease due to dissipation of excess pore water pressures induced by ground movements in clay, or due to evaporation of moisture with subsequent loss of apparent cohesion in silt and fine sand. If ravelling begins within a few minutes of exposure, it is described as fast ravelling, otherwise it is referred to as slow ravelling. Fast ravelling may take place in residual soils and sands with a clay binder below the water table. These materials above the water table are slow ravelling.
3. *Running ground*. In this type of ground, the removal of support from a surface inclined at more than 34° gives rise to a run, the latter occurring until the angle of rest of the material involved is attained. Runs take place in clean, loosely packed gravel, and clean, coarse- to medium-grained sand, both above the water table. In clean, fine-grained, moist sand, a run usually is preceded by ravelling, such behaviour being termed cohesive running.
4. *Flowing ground*. This type of ground moves like a viscous liquid. It can invade a tunnel from any angle and, if not stopped, ultimately fills the excavation. Flowing conditions occur in sands and silts below the water table. Such ground above the water table exhibits either ravelling or running behaviour.
5. *Squeezing ground*. Squeezing ground advances slowly and imperceptibly into a tunnel. There are no signs of fracturing of the sides. Ultimately, the roof may give, and this can produce a subsidence trough at the surface. The two most common reasons why ground squeezes on subsurface excavation are excessive overburden pressure

and the dissipation of residual stress, both eventually leading to failure. Soft and medium clays display squeezing behaviour. Other materials in which squeezing conditions may obtain include shales and highly weathered granites, gneisses and schists.

6. *Swelling ground*. Swelling ground also expands into the excavation but the movement is associated with a considerable volume increase in the ground immediately surrounding the tunnel. Swelling occurs as a result of water migrating into the material of the tunnel perimeter from the surrounding strata. These conditions develop in overconsolidated clays with a plasticity index in excess of about 30% and in certain shales and mudstones, especially those containing montmorillonite. Swelling pressures are of unpredictable magnitude and may be extremely large. For example, the swelling pressure in shallow tunnels may exceed the overburden pressure and it may be as high as 2.0 MPa in overconsolidated clays. The development period may take a few weeks or several months. Immediately after excavation, the pressure is insignificant but the rate of swelling increases after that. In the final stages, the increase slows down.

Boulders within a soft ground matrix may prove difficult to remove, whereas if boulders are embedded in a hard cohesive matrix, they may impede progress and may render a mechanical excavator of almost any type impotent. Large boulders may be difficult to handle unless they are broken apart by jackhammer or blasting.

Water in Tunnels

The amount of water held in a soil or rock mass depends on its reservoir storage properties (see Chapter 4) that, in turn, influence the amount of water that can drain into a tunnel. Isolated heavy flows of water may occur in association with faults, solution pipes and cavities, abandoned mine workings or even pockets of gravel. Tunnels driven under lakes, rivers and other surface bodies of water may tap a considerable volume of flow. Flow also may take place from a perched water table to a tunnel beneath.

Generally, the amount of water flowing into a tunnel decreases as construction progresses. This is due to the gradual exhaustion of water at source and to the decrease in hydraulic gradient, and hence in flow velocity. On the other hand, there may be an increase in flow as construction progresses if construction operations cause fissuring. For instance, blasting may open new water conduits around a tunnel, shift the direction of flow and, in some cases, even cause partial flooding.

Correct estimation of the water inflow into a projected tunnel is of vital importance, as inflow influences the construction programme (Cripps et al., 1989). One of the principal problems created by water entering a tunnel is that of face stability. Secondary problems include removal of excessively wet muck and the placement of a precision-fitted primary lining or of ribs.

The value of the maximum inflow is required and so are the distribution of inflow along the tunnel section and the changes of flow with time. The greatest groundwater hazard in underground work is the presence of unexpected water-bearing zones, and therefore, whenever possible, the position of hydrogeological boundaries should be located. Obviously, the location of the water table, and its possible fluctuations, are of major consequence.

Water pressures are more predictable than water flows as they are nearly always a function of the head of water above the tunnel location. They can be very large, especially in confined aquifers. Hydraulic pressures should be taken into account when considering the thickness of rock that will separate an aquifer from a tunnel. Unfortunately, however, the hydrogeological situation is rarely so easily interpreted as to make accurate quantitative estimates possible.

Sulphate-bearing solutions attack concrete, thus water quality must be investigated. Particular attention should be given to water flowing from sequences containing gypsum and anhydrite. Rocks containing iron pyrite also may give rise to water-carrying sulphates, as well as acidic water.

Most of the serious difficulties encountered during tunnelling operations are directly or indirectly caused by the percolation of water towards the tunnel. As a consequence, most of the techniques for improving ground conditions are directed towards its control. This may be achieved by using drainage, compressed air, grouting or freezing techniques.

Gases in Tunnels

Naturally occurring gas can occupy the pore spaces and voids in rock. This gas may be under pressure, and there have been occasions when gas under pressure has burst into underground workings, causing the rock to fail with explosive force (Bell and Jermy, 2002). Wherever possible the likelihood of gas hazards should be noted during the geological survey, but this is one of the most difficult tunnel hazards to predict. If the flow of gas appears to be fairly continuous, then the entrance to the flow may be sealed with concrete. Often, the supply of gas is exhausted quickly, but cases have been reported where it continued for up to 3 weeks.

Many gases are dangerous. For example, methane, CH_4, which may be encountered in Coal Measures, is lighter than air and can readily migrate from its point of origin. Not only is methane toxic, it also is combustible and highly explosive when 5–15% is mixed with air. Carbon dioxide, CO_2, and carbon monoxide, CO, are both toxic. The former is heavier than air and hangs about the floor of an excavation. Carbon monoxide is slightly lighter than air and as with carbon dioxide and methane, is found in Coal Measures strata. Carbon dioxide also may be associated with volcanic deposits and limestones. Hydrogen sulphide, H_2S, is heavier than air and is highly toxic. It also is explosive when mixed with air. The gas may be generated by the decay of organic substances or by volcanic activity. Hydrogen sulphide may be absorbed by water that then becomes injurious as far as concrete is concerned. Sulphur dioxide, SO_2, is a colourless pungent asphyxiating gas that dissolves readily in water to form sulphuric acid. It usually is associated with volcanic emanations, or it may be formed by the breakdown of pyrite.

Temperatures in Tunnels

Temperatures in tunnels are not usually of concern unless the tunnel is more than 170 m below the surface. When rock is exposed by excavation, the amount of heat liberated depends on the virgin rock temperature, VRT; the thermal properties of the rock; the length of time of exposure; the area, size and shape of exposed rock; the wetness of rock; the air flow rate; the dry bulb temperature; and humidity of the air.

In deep tunnels, high temperatures can make work more difficult. Indeed, high temperatures and rock pressures place limits on the depth of tunnelling. The moisture content of the air in tunnels is always high and, in saturated air, the efficiency of labour declines when the temperature exceeds 25°C, dropping to almost zero when the temperature reaches 35°C. Conditions can be improved by increased ventilation, by water spraying or by using refrigerated air. Air refrigeration is essential when the virgin rock temperature exceeds 40°C.

The rate of increase in rock temperature with depth depends on the geothermal gradient that, in turn, is inversely proportional to the thermal conductivity, k, of the material involved:

$$\text{Geothermal gradient} = \frac{0.05}{k} \text{ (approximately)°C m}^{-1} \tag{9.3}$$

Although the geothermal gradient varies with locality, according to rock type and structure, on average it increases at a rate of 1°C per 30–35 m depth. In geologically stable areas, the mean gradient is 1°C for every 60–80 m, whereas in volcanic districts, it may be as much as 1°C for every 10–15 m depth. The geothermal gradient under mountains is larger than under plains; in the case of valleys, the situation is reversed.

The temperature of the rocks influences the temperature of any water they may contain. Fissure water that flows into workings acts as an efficient carrier of heat. This may be locally more significant than the heat conducted through the rocks themselves. For example, for every litre of water that enters the workings at a virgin rock temperature of 40°C, if the water cools to 25°C before it reaches the pumps, then the heat added to the ventilating air stream will be 62.8 kW.

Earth temperatures can be measured by placing thermometers in drillholes, measurement being taken when a constant temperature is attained. The results, in the form of geoisotherms, can be plotted on the longitudinal section of a tunnel.

Excavation of Tunnels

In soft ground, support is vital and so tunnelling is carried out by using shields. A shield is a cylindrical drum with a cutting edge around the circumference, the cut material being delivered onto a conveyor for removal. The limits of these machines usually are given as an unconfined compressive strength of 20 MPa. Shield tunnelling means that construction can be carried out in one stage at the full tunnel dimension and that the permanent lining is installed immediately after excavation, thereby, providing support as the tunnel is advanced (Fig. 9.12; De Graaf and Bell, 1997).

Figure 9.12

Installation of the segmental lining, Delivery Tunnel North, Lesotho Highlands Water Scheme.

Bentonite slurry is used to support the face in soft ground in a pressure bulkhead machine. This represented a major innovation in mechanized tunnelling, particularly in coarse sediments not suited to compressed air. The bentonite slurry counterbalances the hydrostatic head of groundwater in the soil, and stability is increased further as the bentonite is forced into the pores of the soil, gelling once penetration occurs. The bentonite forms a seal on the surface. However, boulders in soils, such as till, create an almost impossible problem for slurry face machines. A mixed face of hard rock and coarse soil below the water table presents a similar dilemma.

Machine tunnelling in rock uses either a roadheader machine or a tunnel boring machine, TBM. A roadheader generally moves on a tracked base and has a cutting head, usually equipped with drag picks, mounted on a boom (Fig. 9.13). Twin-boom machines have been developed in order to increase the rate of excavation. Roadheaders can cut a range of tunnel shapes and are particularly suited to stratified formations. Some of the heavier roadheaders can excavate massive rocks with unconfined compressive strengths in excess of 100 MPa and even up to 200 MPa. Obviously, the cutting performance is influenced by the presence and character of discontinuities. Basically, excavation by a TBM is accomplished by a cutter head equipped with an array of suitable cutters, which usually is rotated at a constant speed and thrust into the tunnel face by a hydraulic pushing system (Fig. 9.14). The stresses imposed

Figure 9.13

A road header used at Dinorwic Pumped Storage Scheme, Llanberis, North Wales.

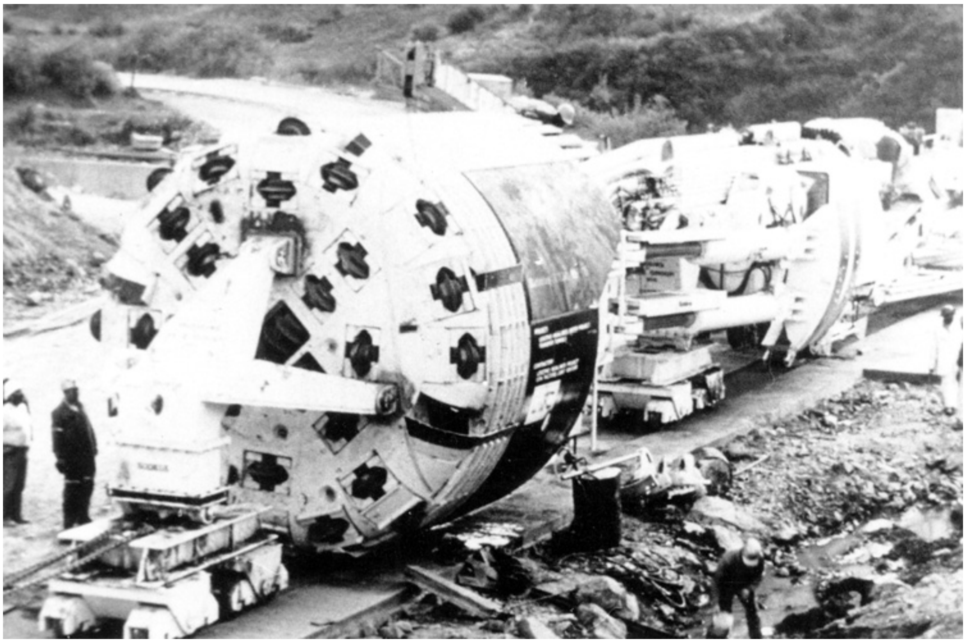


Figure 9.14

One of the tunnel boring machines used for the construction of the Transfer Tunnel, Lesotho Highlands Water Scheme.

on the surrounding rock by a TBM are much less than those produced during blasting and therefore damage to the perimeter is minimized and a sensibly smooth face usually is achieved. What is more, overbreak (see the following text) normally is less during TBM excavation than during drilling and blasting, on average 5% as compared with up to 25% for conventional methods. This means that less support is required. The rate of tunnel drivage obviously is an important economic factor in tunnelling, especially in hard rock. Tunnel boring machines have provided increased rates of advance and so shortened the time taken to complete tunnelling projects. Indeed, they have achieved faster rates of drivage than conventional tunnelling methods in rocks with unconfined compressive strengths of up to 150 MPa. Consequently, tunnels now are excavated much more frequently by TBMs than by conventional drill and blast methods. The unconfined compressive strength commonly is one of the most important properties determining the rate of penetration of a TBM (Castro and Bell, 1995). The rate of penetration in low-strength rocks is affected by problems of roof support and instability, as well as gripper problems. Problems associated with rock masses of high strength are the increased cutter wear and larger thrust (and hence cost) required to induce rock fracture. The rate of penetration also is influenced by the necessity to replace cutters on the head of a TBM, which involves downtime. Cutter wear depends, in part, on the abrasive properties of the rock mass being bored. Whether a rock mass is massive, jointed, fractured, water bearing, weathered or folded, also affect cutter life. For instance, in hard blocky ground some cutters are broken by the tremendous impact loads generated during boring. Moreover, the performance of TBMs is more sensitive to changes in rock properties than conventional drilling and blasting methods, consequently their use in rock masses that have not been thoroughly investigated involves high risk. Hence, the decision to use a machine must be based on a particularly thorough knowledge of the anticipated geological conditions.

Apart from ground stability and support, the most important economic factors in machine tunnelling in hard rock are cutter costs and penetration rate. The rate of wear is basically a function of the abrasive characteristics of the rock mass involved. Penetration rate is a function of cutter geometry, thrust of the machine and the rock strength.

Overbreak refers to the removal of rock material from beyond the payline, the cost of which has to be met by the contractor. Obviously, every effort must be made to keep overbreak to a minimum. The amount of overbreak is influenced by the rock type and discontinuities, as well as the type of excavation.

The conventional method of advancing a tunnel in hard rock is by full-face driving, in which the complete face is drilled and blasted as a unit. However, full-face driving should be used with caution where the rocks are variable. The usual alternatives are the top heading and bench method or the top heading method, whereby the tunnel is worked on an upper and lower section or heading. The sequence of operations in these three methods is illustrated in Figure 9.15.

In tunnel blasting, a cut is opened up approximately in the centre of the face in order to provide a cavity into which subsequent shots can blast. Delay detonation refers to a face being fired with the shots being detonated in a predetermined sequence. The first shots in the round blast out the cut, and subsequent shots blast in sequence to form the free face.

Drilling and blasting can damage the rock structure, depending on the properties of the rock mass and the blasting technique. As far as technique is concerned, attention should be given to the need to maintain adequate depths of pull, to minimize overbreak and to maintain blasting vibrations below acceptable levels. The stability of a tunnel roof in fissured rocks depends on the formation of a natural arch, and this is influenced by the extent of disturbance, the irregularities of the profile and the relationship between tunnel size and fracture pattern. The amount of overbreak tends to increase with increased depths of pull since drilling inaccuracies are magnified. In such situations, not only does the degree of overbreak become very expensive in terms of grout and concrete backfill but it may give rise to support problems and subsidence over the crown of the tunnel. However, overbreak can be reduced by accurate drilling and a carefully controlled scale of blasting. Controlled blasting may be achieved either by presplitting the face to the desired contour or by smooth blasting.

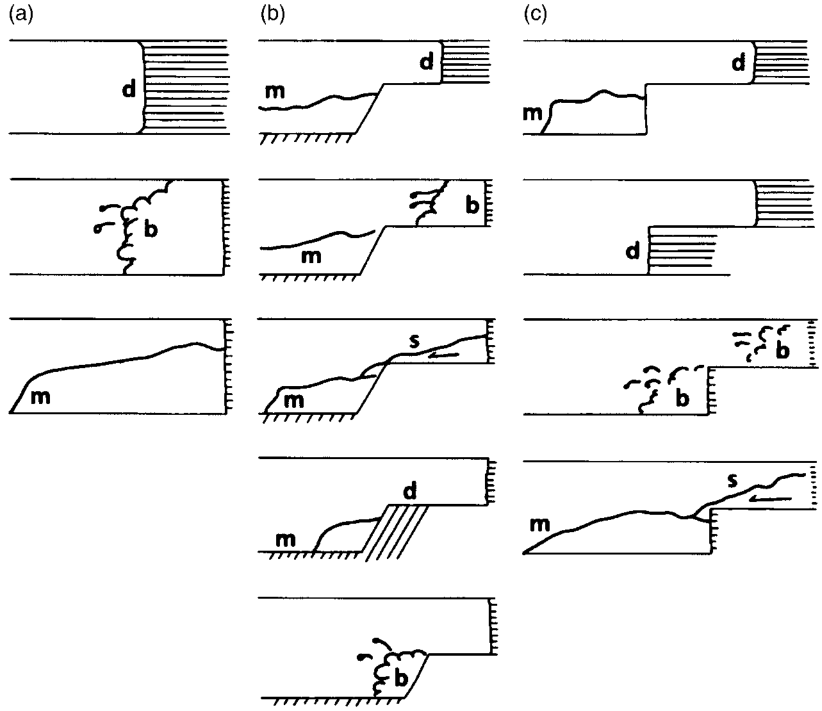


Figure 9.15

Tunnelling by drilling and blasting, (a) Full-face, (b) top heading and bench and (c) top heading. Bench drilled horizontally. Phases: d = drilling; b = blasting; m = mucking; s = scrapping.

In the pre-splitting method, a series of holes is drilled around the perimeter of the tunnel, loaded with explosives that have a low charging density and detonated before the main blast. The initial blast develops a fracture that spreads between the holes. Hence, the main blast leaves an accurate profile. The technique is not particularly suited to slates and schists because of their respective cleavage and schistosity, Indeed, slates tend to split along, rather than across their cleavage. Although it is possible to presplit jointed rock masses adequately, the tunnel profile still is influenced by the pattern of the jointing.

Smooth blasting has proved a more successful technique than presplitting. Here again explosives with a low charging density are used in closely spaced perimeter holes. For example, the ratio between burden and hole spacing usually is 1:0.8, which means that crack formation is controlled between the drillholes and hence is concentrated within the final contour. The holes are fired after the main blast, their purpose being to break away the last fillet of rock between the main blast and the perimeter. Smooth blasting cannot be carried out without good drilling precision. Normally, smooth blasting is restricted to the roof and walls of a tunnel but occasionally it is used in the excavation of the floor. Because fewer cracks are produced in the surrounding rock, it is stronger and so the desired roof curvature can be maintained to the greatest possible extent. Hence, the load-carrying capacity of the rock mass is utilized properly.

Analysis of Tunnel Support

The time a rock mass may remain unsupported in a tunnel is called its stand-up time or bridging capacity. This mainly depends on the magnitude of the stresses within the unsupported rock mass, which in their turn depend on its span, strength and discontinuity pattern. If the bridging capacity of the rock is high, the rock material next to the heading will stay in place for a considerable time. In contrast, if the bridging capacity is low, the rock will immediately start to fall at the heading so that supports have to be erected as soon as possible.

The primary support for a tunnel in rock masses excavated by drilling and blasting, in particular, may be provided by rock bolts (with or without reinforcing wire mesh), shotcrete or steel arches (Clough, 1981). Rock bolts maintain the stability of an opening by suspending the dead weight of a slab from the rock above; by providing a normal stress on the rock surface to clamp discontinuities together and develop beam action; by providing a confining pressure to increase shearing resistance and develop arch action; and by preventing key blocks from becoming loosened so that the strength and integrity of the rock mass is maintained. Shotcrete can be used for lining tunnels. For example, a layer 150 mm thick, around a tunnel 10 m in diameter, can safely carry a load of 500 kPa, corresponding to a burden of approximately 23 m of rock, more than has ever been observed with rock falls. When combined with rock bolting and reinforcing wire mesh, shotcrete has proved an excellent temporary support for all qualities of rock. In very bad cases, steel arches can be used for reinforcement of weaker tunnel sections.

A classification of rock masses is of primary importance in relation to the design of the type of tunnel support. Lauffer's (1958) classification represented an appreciable advance in the art of tunnelling since it introduced the concept of an active unsupported rock span and the corresponding stand-up time, both of which are very relevant parameters for determination of the type and amount of primary support in tunnels. The active span is the width of the tunnel or the distance from support to the face in cases where this is less than the width of the tunnel. The relationships found by Lauffer are given in Figure 9.16.

Bieniawski (1974, 1989) maintained that the uniaxial compressive strength of rock material; the rock quality designation; the spacing, orientation and condition of the discontinuities; and groundwater inflow were the factors that should be considered in any engineering classification

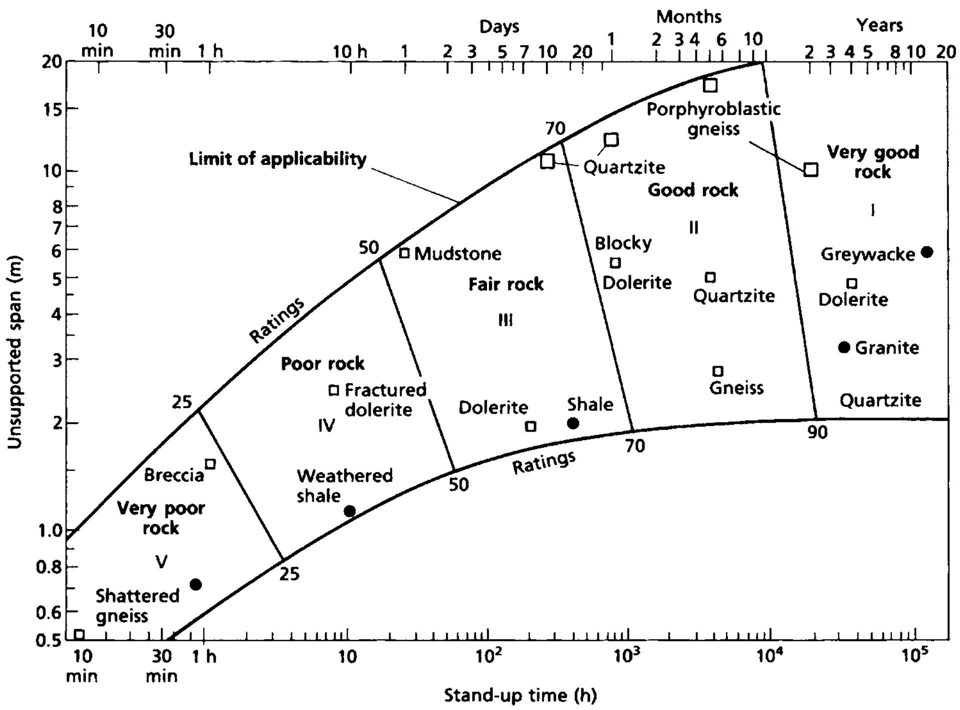


Figure 9.16

Geomechanics classification of rock masses for tunnelling. South African case studies are indicated by squares, whereas those from Alpine countries are shown by dots (after Lauffer, 1958). With kind permission of Springer.

of rock masses. A classification of rock masses based on these parameters is given in Table 9.4. Each parameter is grouped into five categories, and the categories, are given a rating. Once determined, the ratings of the individual parameters are summed to give the total rating or class of the rock mass. The higher the total rating, the better are the rock mass conditions. However, the accuracy of the rock mass rating, RMR, in certain situations may be open to question. For example, it does not take into account the effects of blasting on rock masses. Neither does it consider the influence of in situ stress on stand-up time nor the durability of the rock. The latter can be assessed in terms of the geodurability classification (see Chapter 3). Nevertheless, Dalgic (2002) suggested that an estimate of the support pressure, P_i, could be obtained from the RMR as follows:

$$P_i = (100 - RMR)\,\gamma_b B/100 \tag{9.4}$$

where γ_b is the bulk density of the rock and B is the opening of the excavation.

Suitable support measures at times must be adopted to attain a stand-up time longer than that indicated by the total rating or class of the rock mass. These measures constitute the

Table 9.4. The rock mass rating system (geomechanics classification of rock masses) (after Bieniawski, 1989). With kind permission of Wiley

(a) Classification parameters and their ratings

	Parameter		**Range of values**				
1	Strength of Intact rock material	Point-load strength Index (MPa)	>10	4–10	2–4	1–2	For this low range, uniaxial compressive test is preferred
		Uniaxial compressive strength (MPa)	>250	100–250	50–100	25–50	5–25 1–5 <1
	Rating		15	12	7	4	2 1 0
2	Drill core quality RQD (%)		90–100	75–90	50–75	25–50	<25
	Rating		20	17	13	8	3
3	Spacing of discontinuities		>2 m	0.6–2 m	200–600 mm	60–200 mm	<60 mm
	Rating		20	15	10	8	5
4	Condition of discontinuities		Very rough surfaces Not continous No separation Unweathered wall rock	Slightly rough surfaces Separation <1 mm Slightly weathered walls	Slightly rough surfaces Separation <1 mm Highly weathered walls	Slickensided surfaces or gouge <5 mm thick or separation 1–5 mm continuous	Soft gouge >5 mm thick or separation >5 mm continuous
	Rating		30	25	20	10	0
5	Groundwater	Inflow per 10 m tunnel length (1/min)	None	<10	10–25	25–125	>125
			or	or	or	or	or
		Ratio $\frac{\text{Joint water pressure}}{\text{Major principle stess}}$	0	<0.1	0.1–0.2	0.2–0.5	>0.5
			or	or	or	or	or
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating		15	10	7	4	0

Continued

Table 9.4. The rock mass rating system (geomechanics classification of rock masses) (after Bieniawski, 1989)—cont'd

(b) Rating adjustment for discontinuity orientations

Parameter	**Range of values**				
Strike and dip orientations of discontinuities	Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings: Tunnels and mines	0	–2	–5	–10	–12
Foundations	0	–2	–7	–15	–25
Slopes	0	–5	–25	–50	–60

(c) Rock mass classes determined from total ratings

Rating	100←81	80←61	60←41	40←21	<20
Class no.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

(d) Meaning of rock mass classes

Class no.	I	II	III	IV	V
Average stand-up time	20 yr for 15 m span	1 yr for 10 m span	1 wk for 5 m span	10 h for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	>400	300–400	200–300	100–200	<100
Friction angle of the rock mass (deg.)	>45	35–45	25–35	15–25	<15

Table 9.5. Guide for the selection of primary support at shallow depth, size 5–15 m, construction by drilling and blasting (after Bieniawski, 1974). With kind permission of Balkema

	Alternative support systems		
Rock mass class	**Mainly rock bolts (20 mm diameter, length half of tunnel width, resin bonded)**	**Mainly shotcrete**	**Mainly steel ribs**
I	Generally no support required		
II	Rock bolts spaced 1.5–2 m, plus occasional wire mesh	Shotcrete 50 mm in crown.	Uneconomic
III	Rock bolts spaced 1.0–1.5 m plus wire mesh and 30 mm shotcrete in crown where required	Shotcrete 100 mm in crown and 50 mm on sides, plus occasional wire mesh and rock bolts where required	Light sets spaced 1.5–2 m
IV	Rock bolts spaced 0.5–1.0 m plus wire mesh and 30–50 mm shotcrete in crown and sides	Shotcrete 150 mm in crown and 100 mm on sides plus wire mesh and rock bolts, 3 m long and spaced 1.5 m	Medium sets spaced 0.7–1.5 m plus 50 mm shotcrete in crown and sides.
V	Not recommended	Shotcrete 200 mm in crown and 150 mm on sides plus wire mesh, rock bolts and light steel sets. Seal face close invert	Heavy sets spaced 0.7 m with lagging. Shotcrete 80 mm thick to be applied immediately after blasting

primary or temporary support. Their purpose is to ensure tunnel stability until the secondary or permanent support system, for example, a concrete lining, is installed. The form of primary support depends on depth below the surface, tunnel size and shape, and method of excavation. Table 9.5 indicates the primary support measures for shallow tunnels 5–12 m in diameter driven by drilling and blasting.

Barton et al. (1975) pointed out that Bieniawski (1974), in his analysis of tunnel support, more or less ignored the roughness of joints, the frictional strength of the joint fillings and the rock load. They, therefore, proposed the concept of rock mass quality, *Q*, which could be used as a means of rock classification for tunnel support (Table 9.6). They defined the rock mass quality in terms of six parameters:

1. The RQD or an equivalent estimate of joint density.
2. The number of joint sets, J_n, which is an important indication of the degree of freedom of a rock mass. The RQD and the number of joint sets provide a crude measure of relative block size.

Table 9.6. Classification of individual parameters used in the Norwegian Geotechnical Institute (NGI) tunnelling quality index or "*Q*" system (after Barton et al., 1975). With kind permission of the NGI

Description	Value	Notes
1. ***Rock quality designation***	RQD	1. Where RQD is reported or measured as ≤10 (including 0), a nominal value of 10 is used to evaluate *Q*
A Very poor	0–25	2. RQD intervals of 5, i.e. 100, 95, 90, etc. are sufficiently accurate
B Poor	25–50	
C Fair	50–75	
D Good	75–90	
E Excellent	90–100	
2. ***Joint set number***	J_n	1. For intersections, use $(3.0 \times J_n)$
A Massive, no or few joints	0.5–1.0	2. For portals, use $(2.0 \times J_n)$
B One joint set	2	
C One joint set plus random	3	
D Two joint sets	4	
E Two joint sets plus random	6	
F Three joint sets	9	
G Three joint sets plus random	12	
H Four or more joint sets, random, heavily joined "sugar cube," etc.	15	
J Crushed rock, earthlike	20	
3. ***Joint roughness number***	J_r	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
(a) Rock wall contact and		
(b) Rock wall contact before 10 cm shear		2. $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength
A Discontinuous joints	4	
B Rough or irregular, undulating	3	
C Smooth, undulating	2	
D Slickensided, undulating	1.5	
E Rough or irregular, undulating	1.5	
F Smooth, planar	1.0	
G Slickensided, planar	0.5	
(c) No rock wall contact when sheared		

H Zone containing clay minerals, thick enough to prevent rock wall contact	1.0		
I Sandy, gravelly or crushed zone, thick enough to prevent rock wall contact	1.0		
4. ***Joint alteration number***	J_a	Φ_r (approx.)	1. Values of Φ_r, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present
(a) Rock wall contact			
A Tightly healed, hard non-softening, impermeable filling	0.75	–	
B Unaltered joint walls, surface staining only	1.0	(25–35°)	
C Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	2.0	(25–30°)	
D Silty, or sandy clay coatings, small clay-fraction (non-softening)	3.0	(20–25°)	
E Softening or low-fraction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum, and graphite, etc. and small quantities of swelling clays (discontinuous coatings, 1–2 mm or less in thickness).	4.0	(8–16°)	
(b) Rock wall contact before 10 cm shear			
F Sandy particles, clay-free disintegrated rock, etc.	4.0	(25–30°)	
G Strongly overconsolidated, non-softening clay mineral fillings (continuous, < 5 mm thick)	6.0	(16–24°)	
H Medium or low overconsolidation, softening, clay mineral fillings (continuous, < 5 mm thick)	8.0	(12–16°)	

Continued

Table 9.6. Classification of individual parameters used in the Norwegian Geotechnical Institute (NGI) tunnelling quality index or ''*Q*'' system (after Barton et al., 1975)—cont'd

Description	**Value**		**Notes**
J Swelling clay fillings, i.e. montmorillonite (continuous < 5 mm thick). Values of J_a depend on percentage of swelling clay-size particles, and access to water	8.0–12.0	(6–12°)	
(c) No rock wall contact when sheared			
K Zones or bands of disintegrated or crushed rock and clay	6.0		
L (see G, H, and J for clay, conditions)	8.0		
M	8.0–12.0	(6–24°)	
N Zones or bands of silty or sandy clay, small clay fraction (non-softening)	5.0		
O Thick, continuous zones or bands of clay (see G, H and J for clay conditions)	10.0–13.0		
R	13.0–20.0	(6–24°)	
5. ***Joint water reduction factor***	J_w	Approx. water pressure (kgf/cm^2)	1. Factors C to F are crude estimates. Increase J_w if drainage measures are installed 2. Special problems caused by ice formation are not considered
A Dry excavations or minor inflow, i.e. < 5 l/min locally	1.0	< 1.0	
B Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0–2.5	
C Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5–10.0	
D Large inflow or high pressure, considerable outwash of fillings	0.33	2.5–10.0	
E Exceptionally high inflow or pressure at blasting, decaying with time	0.2–0.1	> 10	

F Exceptionally high inflow or pressure continuing without decay	0.1–0.05	>10		
6. ***Stress reduction factor***	SRF			1. Reduce these values of SRF by 25–50% if the relevant shear zones only influence but do not intersect the excavation 2. For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$, and $0.6\sigma_t$ where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses 3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).
(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				
A Multiple occurrences of weakness zones containing clay or chemically distintegrated rock, very loose surrounding rock (any depth)	10.0			
B Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50 m)	5.0			
C Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50 m)	2.5			
D Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5			
E Single shear zones in competent rock (clay-free) (depth of excavation < 50 m)	5.0			
F Single shear zones in competent rock (clay-free), (depth of excavation > 50 m)	2.5			
G Loose open joints, heavily jointed or "sugar cube" (any depth)	5.0			
(b) Competent rock, rock-stress problems	σ_c/σ_1	σ_t/σ_1	SRF	
H Low stress, near surface	>200	>13	2.5	
J Medium stress	200–10	13–0.66	1.0	
K High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10–5	0.66–0.33	0.5–2	

Continued

Table 9.6. Classification of individual parameters used in the Norwegian Geotechnical Institute (NGI) tunnelling quality index or "*Q*" system (after Barton et al., 1975)—cont'd

Description	Value		SRF
L Mild rock burst (massive rock)	5–2.5	0.33–0.16	5–10
M Heavy rock burst (massive rock)	< 2.5	< 0.16	10–20
(c) Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure			
N Mild squeezing-rock pressure			5–10
O Heavy squeezing-rock pressure			10–20
(d) Swelling rock, chemical swelling activity depending on presence of water			
P Mild swelling-rock pressure			5–10
R Heavy swelling-rock pressure			10–20

Additional notes on the use of these tables

When making estimates of the rock mass quality (*Q*) the following guidelines should be followed, in addition to the notes listed in the tables:

1. When drillhole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay-free rock masses: RQD = 115 – 3.3 J_v. (approx.) where J_v = total number of joints per m^3 (RQD = 100 for $J_v < 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage, or bedding. etc. If strongly developed these parallel joints should obviously be counted as a complete joint set. However, if there are few joints visible, or only occasional breaks in the core due to these features, then it will he more appropriate to count them as "random" when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay-filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluatitig *Q*. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to present or future *in situ* conditions. A very conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

3. The roughness of the most unfavourable joint set, J_r. The joint roughness and the number of joint sets determine the dilatancy of the rock mass.
4. The degree of alteration or filling of the most unfavourable joint set, J_a. The roughness and degree of alteration of the joint walls or filling materials provide an approximation of the shear strength of the rock mass.
5. The degree of water seepage, J_w.
6. The stress reduction factor, SRF, which accounts for the loading on a tunnel caused either by loosening loads in the case of clay-bearing rock masses, or unfavourable stress–strength ratios in the case of massive rock. Squeezing and swelling also are taken account of in the SRF.

They provided a rock mass description and ratings for each of the six parameters that enabled the rock mass quality, Q, to be derived from:

$$Q = \frac{\mathrm{RQD}}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\mathrm{SRF}} \tag{9.5}$$

The numerical value of Q ranges from 0.001 for exceptionally poor-quality squeezing ground, to 1000 for exceptionally good-quality rock (practically unjointed). Rock mass quality, together with the support pressure and the dimensions and purpose of the underground excavation are used to estimate the type of suitable permanent support. A fourfold change in rock mass quality indicates the need for a different support system. Zones of different rock mass quality are mapped and classified separately. However, in variable conditions where different zones occur within a tunnel, each for only a few metres, it is more economic to map the overall quality and to estimate an average value of rock mass quality, from which a design of a compromise support system can be made (Barton, 1988).

The Q value is related to the type and amount of support by deriving the equivalent dimensions of the excavation. The latter is related to the size and purpose of the excavation, and is obtained from:

$$\text{Equivalent dimension} = \frac{\text{Span or height of wall}}{\text{ESR}} \tag{9.6}$$

where ESR is the excavation support ratio related to the use of the excavation and the degree of safety required. Some values of ESR are shown in Table 9.7.

Stacey and Page (1986) made use of the Q system to develop design charts to determine the factor of safety for unsupported excavations, the spacing of rock bolts over the face of an

Table 9.7. Equivalent support ratio for different excavations

Excavation category	ESR
1. Temporary mine openings	3–5
2. Vertical shafts	
Circular section	2.5
Rectangular/square section	2.0
3. Permanent mine openings, water tunnels for hydropower (excluding high-pressure penstocks), pilot tunnels, drifts and headings for large excavations	1.6
4. Storage caverns, water treatment plants, minor highway and railroad tunnels, surge chambers, access tunnels	1.4
5. Power stations, major highway or railroad tunnels, civil defence chambers, portals, intersections	1.0
6. Underground nuclear power stations, railroad, stations, factories	0.8

excavation and the thickness of shotcrete on an excavation (Figs. 9.17a, b and c, respectively). For civil engineering applications, a factor of safety exceeding 1.2 is required if the omission of support is to be considered. The support values suggested in the charts are for primary support. The values should be doubled for long-term support.

Underground Caverns

The site investigation for an underground cavern has to locate a sufficiently large mass of sound rock in which the cavern can be excavated. Because caverns usually are located at appreciable depth below ground surface, the rock mass often is beneath the influence of weathering and consequently the chief considerations are rock quality, geological structure and groundwater conditions. The orientation of an underground cavern usually is based on an analysis of the joint pattern, including the character of the different joint systems in the area and, where relevant, also on the basis of the stress distribution. It normally is considered necessary to avoid an orientation whereby the long axis of a caverrn is parallel to steeply inclined major joint sets (Hoek and Brown, 1980). Wherever possible, caverns should be orientated so that fault zones are avoided.

Displacement data provide a direct means of evaluating cavern stability. Displacements that have exceeded the predicted elastic displacements by a factor of 5–10 generally have resulted in decisions to modify support and excavation methods. In a creep-sensitive material, such as may occur in a major shear zone or zone of soft altered rock, the natural stresses concentrated around an opening cause time-dependent displacements that, if restrained by support, result in a build-up of stress on the support. Conversely, if a rock mass is not sensitive to creep, stresses around an opening normally are relieved as blocks displace towards

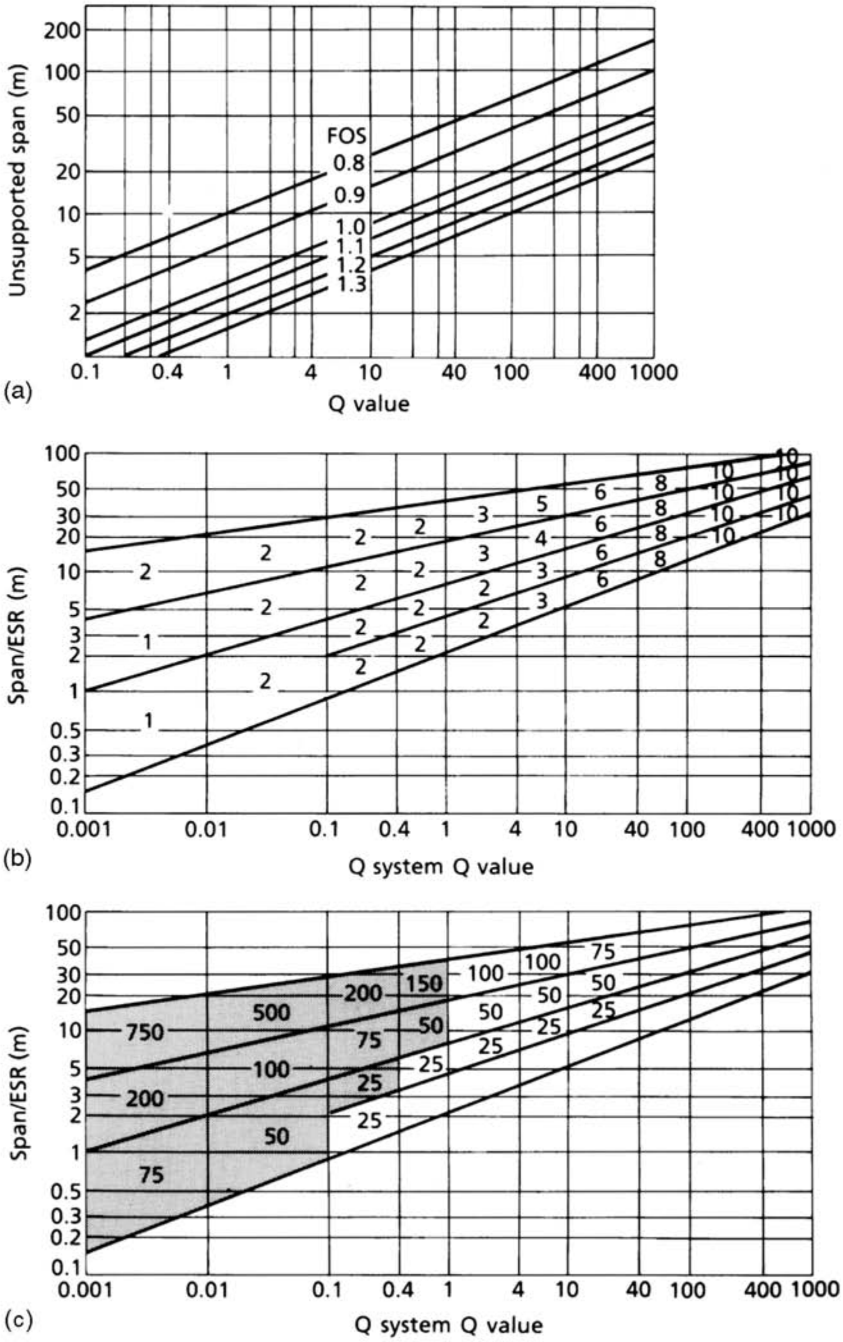


Figure 9.17

(a) Relationship between unsupported span and *Q* value (b) Bolt spacing estimation using the *Q* system, bolt spacing – m^2 of excavation per bolt, where the area per bolt is greater than 6 m^2, spot bolting is implied. (c) Shotcrete and wire mesh support estimation using the *Q* system. Thickness of shotcrete in millimetres (mesh reinforcement in the shaded areas). Note that the very thick applications of shotcrete are not practical but values are included for completeness (after Stacey and Page, 1986). The support intensity in design charts (b) and (c) is appropriate for primary support; where long-term support is required, the design chart values should be modified as follows: (i) divide area per bolt by 2; (ii) multiple shotcrete thickness by 2. FOS = factor of safety.

the opening. However, initial movements may be influenced by the natural stresses concentrated around an opening and under certain boundary conditions may continue to act even after large displacements have occurred.

The angle of friction for tight irregular joint surfaces commonly is greater than 45° and, as a consequence, the included angle of any wedge opening into the roof of a cavern has to be 90° or more, if the wedge is to move into the cavern. A tight rough joint system therefore only presents a problem when it intersects the surface of a cavern at relatively small angles or is parallel to the surface of the cavern. However, if material occupying a thick shear zone has been reduced to its residual strength, then the angle of friction could be as low as 15° and, in such an instance, the included angle of a wedge would be 30°. Such a situation would give rise to a very deep wedge that could move into a cavern. Displacement of wedges into a cavern is enhanced if the ratio of the intact unconfined compressive strength to the natural stresses concentrated around the cavern is low. Values of less than 5 are indicative of stress conditions in which new extension fractures develop about a cavern during its excavation. Wedge failures are facilitated by shearing and crushing of the asperities along discontinuities as wedges are displaced.

The walls of a cavern may be influenced by the prevailing state of stress, especially if the tangential stresses concentrated around the cavern approach the intact compressive strength of the rock (Gercek and Genis, 1999). In such cases, extension fractures develop near the surface of the cavern as it is excavated and cracks produced by blast damage become more pronounced. The problem is accentuated if any lineation structures or discontinuities run parallel with the walls of the cavern. Indeed, popping of slabs of rock may take place from cavern walls.

Rock bursts have occurred in underground caverns at rather shallow depths, particularly where they were excavated in the sides of valleys and on the inside of faults when the individual fault passed through a cavern and dipped towards an adjacent valley. Bursting can take place at depths of 200–300 m, when the tensile strength of the rocks varies between 3 and 4 MPa.

Three methods of blasting normally are used to excavate underground caverns (Fig. 9.18). Firstly, in the overhead tunnel, the entire profile is drilled and blasted together or in parts by horizontal holes. Secondly, benching with horizontal drilling may be used to excavate the central parts of a large cavern. Thirdly, the bottom of a cavern may be excavated by benching, the blastholes being drilled vertically. The central part of a cavern also can be excavated by vertical benching, provided that the upper part has been excavated to a sufficient height or that the walls of the cavern are inclined. Indeed, once the crown of a cavern has been excavated, it may be more economical to excavate the walls in a series of large deep bench cuts exposing substantial areas of wall in a single blast. Under these conditions, however,

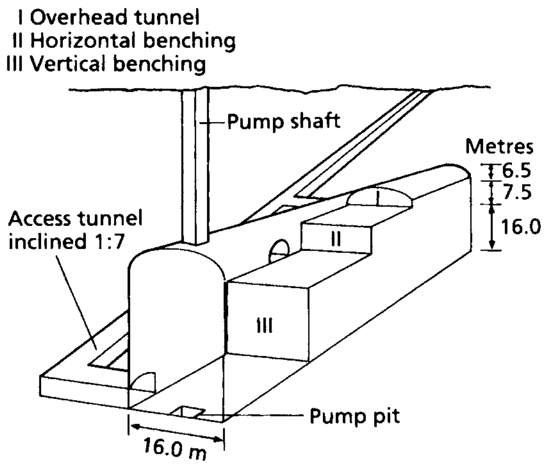


Figure 9.18

Main stages in the excavation of underground caverns.

an unstable wedge can be exposed and fail before it is supported. Smooth blasting is used to minimize fragmentation in the surrounding rock.

The support pressures required to maintain the stability of a cavern increase as its span increases so that for larger caverns, standard-sized bolts arranged in normal patterns may not be sufficient to hold the rock in place. Most caverns have arched crowns with span to rise ratios, B/R, of 2.5–5.0. In general, higher support pressures are required for flatter roofs. Frequently, the upper parts of the walls are more heavily bolted in order to help support the haunches and the roof arch, whereas the lower walls may be either slightly bolted or even unbolted (Hoek et al., 1995).

Shafts and Raises

The geological investigation prior to shaft sinking or raise construction should provide detailed information relating to the character of the ground conditions. The hydrogeological conditions, that is, position of the water table, hydrostatic pressures, especially artesian pressures, location of inflow and its quantification, as well as chemical composition of the water, are obviously of paramount importance. Indeed, groundwater inflow from deep aquifers presents a major hazard in shaft sinking operations since hydrostatic pressure increases with depth. The investigation also should provide detailed information relating to the character of the rocks involved, noting, where appropriate, their fracture index, strength, porosity and permeability. The data obtained should enable the best method of construction and the design of the lining to be made. In addition, the data should indicate where any ground stability and groundwater control measures are needed. Hence, a drillhole should be cored along the full length of the centre-line of a shaft and appropriate down-hole tests conducted.

According to Auld (1992), the best shape of a shaft lining to resist loads imposed by rock deformation and hydrostatic pressure, particularly one that is deep and has a large cross-sectional area, is circular since the induced stresses in the lining are all compressional. The circular shape minimizes the effect of inward radial closure exhibited by rock masses under high overburden stresses. In fact, no pressure is applied to the lining of a shaft in strong competent rock masses as inward deformation is minimal, being compatible with the elastic properties of the rock. Consequently, the design of shaft linings normally does not allow for rock loading when shafts are sunk in strong competent rock masses. However, the hydrostatic pressures must be taken into account in shaft design.

A shaft is driven vertically downwards, whereas a raise is driven either vertically or at steep angle upwards. Excavation is either by drilling and blasting or by a shaft boring machine. In shaft sinking, drilling usually is easier than in tunnelling but blasting is against gravity and mucking is slow and therefore expensive. In contrast, mucking and blasting are simpler in raising operations but drilling is difficult. A raise is of small cross-sectional area, if the excavation is to have a large diameter, then enlargement is done from above, the primary raise excavation being used as a muck shute. Where a raise emerges at the surface through unconsolidated material, this section is excavated from the surface. Shaft boring machines can either excavate a shaft downwards to its required diameter in a single pass or a small diameter shaft can be excavated that subsequently is reamed, from the top or bottom, to the required size. Rock bolts and wire mesh or rock bolts, wire mesh and shotcrete may be used for temporary support. Liner plates, pre-cast concrete segments or fabricated steel tubbing may be used for temporary support in weak ground conditions. The temporary lining is installed as shaft sinking proceeds. The permanent lining is constructed from the top down, however, if temporary support has been used, then the permanent lining may be installed from the bottom up.

A shaft usually will sink through a series of different rock types, and the two principal problems likely to be encountered are varying stability of the walls and ingress of water. Indeed, these two problems frequently occur together, and they are likely to be met with in rock masses with a high fracture index, weak zones being particularly hazardous. They are most serious, however, in unconsolidated deposits, especially loosely packed gravels, sands and silts. The position of the water table is highly significant, as is the pore water pressure.

A simple and effective method of dealing with groundwater in shaft sinking is to pump from a sump within the shaft. However, problems arise when the quantity pumped is so large that the rate of inflow under high head causes instability in the sides of the shaft or prevents the fixing and back-grouting of the shaft lining. Although the idea of surrounding a shaft by a ring of bored wells is at first sight attractive, there are practical difficulties in achieving effective lowering of the water table. In fissured rock masses or variable water bearing soils, there is a tendency for the water flow to by-pass the wells and take preferential paths directly into the excavation.

Where the stability of the wall and/or the ingress of water are likely to present problems, one of the most frequent techniques resorted to in shaft sinking is ground freezing. Freezing transforms weak waterlogged materials into ones that are self-supporting and impervious. Therefore, it affords temporary support to an excavation as well as being a means of excluding groundwater. Normally, the freeze probes are laid out in linear fashion so that an adequate boundary wall encloses the future excavation when the radial development of ice about each probe unites to form a continuous section of frozen ground. The usual coolant is brine, although liquid nitrogen is used in special circumstances.

Ground freezing means heat transfer and, in the absence of moving groundwater, this is brought about by conduction. Thus, the thermal conductivities of the materials involved govern the rate at which freezing proceeds. These values fall within quite narrow limits for all types of frozen ground. This is why ground freezing is a versatile technique and can deal with a variety of soil and rock types in a stratal sequence. But a limitation is placed on the freezing process by the unidirectional flow of groundwater. For a brine-freezing project, a velocity exceeding 2 m per day will seriously affect and distort the growth of an ice wall. The tolerance is much wider when liquid nitrogen is used.

Grouting can be an economical method of eliminating or reducing the flow of groundwater into shafts if the soil or rock conditions are suitable for accepting cement or chemical grouts. This is because the perimeter of the grouted zone is relatively small in relation to the depth of the excavation.

Reservoirs

There are a range of factors that influence the feasibility and economics of a proposed reservoir site. The most important of these is generally the location of the dam. After that, consideration must be given to the run-off characteristics of the catchment area, the watertightness of the proposed reservoir basin, the stability of the valley sides, the likely rate of sedimentation in the new reservoir, the quality of the water and, if it is to be a very large reservoir, the possibility of associated seismic activity. Once these factors have been assessed, they must be weighed against the present land use and social factors. The purposes that the reservoir will serve must also be taken into account in such a survey.

Although most reservoirs today serve multiple purposes, their principal function, no matter what their size, is to stabilize the flow of water, firstly, to satisfy a varying demand from consumers and, secondly, to regulate water supplied to a river course. In other words, water is stored at times of excess flow to conserve it for later release at times of low flow, or to reduce flood damage downstream.

The most important physical characteristic of a reservoir is its storage capacity. Probably the most important aspect of storage in reservoir design is the relationship between capacity and yield. The yield is the quantity of water that a reservoir can supply at any given time. The maximum possible yield equals the mean inflow less evaporation and seepage loss. In any consideration of yield, the maximum quantity of water that can be supplied during a critical dry period (i.e. during the lowest natural flow on record) is of prime importance and is defined as the safe yield.

The maximum elevation to which the water in a reservoir basin rises during ordinary operating conditions is referred to as the top water or normal pool level. For most reservoirs, this is fixed by the top of the spillway. Conversely, minimum pool level is the lowest elevation to which the water is drawn under normal conditions, this being determined by the lowest outlet. Between these two levels, the storage volume is termed the useful storage, whereas the water below the minimum pool level, because it cannot be drawn upon, is the dead storage. During floods, the water level may rise above top water level but this surcharge cannot be retained since it is above the elevation of the spillway.

Problems may emerge both upstream and downstream in any adjustment of a river regime to the new conditions imposed by a reservoir. Deposition around the head of a reservoir may cause serious aggradation upstream, resulting in a reduced capacity of the stream channels to contain flow. Hence, flooding becomes more frequent, and the water table rises. Removal of sediment from the outflow of a reservoir can lead to erosion in the river regime downstream of the dam, with consequent acceleration of headward erosion in tributaries and lowering of the water table.

Investigation of Reservoir Sites

In an investigation of a proposed reservoir site, consideration must be given to the amount of rainfall, run-off, infiltration and evapotranspiration that occurs in the catchment area, as well as to the geological conditions. The climatic and topographical conditions therefore are important, as is the type of vegetative cover. Accordingly, the two essential types of basic data needed for reservoir design studies are adequate topographical maps and hydrological records. Indeed, the location of a large impounding direct supply reservoir is influenced very much by topography since this governs its storage capacity. Initial estimates of storage capacity can be made from topographic maps or aerial photographs, more accurate information being obtained, where necessary, from subsequent surveying. Catchment areas and drainage densities can also be determined from maps and airphotos.

Reservoir volume can be estimated, firstly, by planimetering areas upstream of the dam site for successive contours up to proposed top water level. Secondly, the area between two successive contours is multiplied by the contour interval to give the interval volume, the summation of the interval volumes providing the total volume of the reservoir site.

Records of stream flow are required for determining the amount of water available for storage purposes. Such records contain flood peaks and volumes that are used to determine the amount of storage needed to control floods, and to design spillways and other outlets. Records of rainfall are used to supplement stream flow records or as a basis for computing stream flow where there are no flow records obtainable. Losses due to seepage and evaporation also must be taken into account.

The field reconnaissance provides indications of the areas where detailed geological mapping may be required and where to locate drillholes, such as in low narrow saddles or other seemingly critical areas in the reservoir rim. Drillholes on the flanks of reservoirs should be drilled at least to the proposed floor level. Permeability and pore water tests can be carried out in these drillholes.

Leakage from Reservoirs

The most attractive site for a large impounding reservoir is a valley constricted by a gorge at its outfall with steep banks upstream so that a small dam can impound a large volume of water with a minimum extent of water spread. However, two other factors have to be taken into consideration, namely, the watertightness of the basin and bank stability. The question of whether or not significant water loss will take place is determined chiefly by the groundwater conditions, more specifically by the hydraulic gradient. Accordingly, once the groundwater conditions have been investigated, an assessment can be made of watertightness and possible groundwater control measures.

Leakage from a reservoir takes the form of sudden increases in stream flow downstream of the dam site with boils in the river and the appearance of springs on the valley sides. It may be associated with major defects in the geological structure, such as solution channels, fault zones or buried channels through which large and essentially localized flows take place. Seepage is a more discreet flow, spread out over a larger area but may be no less in total amount.

The economics of reservoir leakage vary. Although a leaky reservoir may be acceptable in an area where run-off is distributed evenly throughout the year, a reservoir basin with the same rate of water loss may be of little value in an area where run-off is seasonally deficient. A river-regulating scheme can operate satisfactorily despite some leakage from a reservoir, and reservoirs used largely for flood control may be effective even if they are very leaky. In contrast, leakage from a pumped storage reservoir must be assessed against pumping costs.

Serious water loss has led, in some instances, to the abandonment of reservoirs. Such examples include the Jerome Reservoir in Idaho, the Cedar Reservoir in Washington, the Monte Jacques Reservoir in Spain, the Hales Bar Reservoir in Tennessee and the Hondo Reservoir in New York.

Apart from the conditions in the immediate vicinity of the dam, the two factors that determine the retention of water in reservoir basins are the piezometric conditions in, and the natural permeability of, the floor and flanks of the basin. Knill (1971) pointed out that four groundwater conditions existed on the flanks of a reservoir, namely:

1. The groundwater divide and piezometric level are at a higher elevation than that of the proposed top water level. In this situation, no significant water loss takes place.
2. The groundwater divide, but not the piezometric level, is above the top water level of the reservoir. In these circumstances, seepage can take place through a separating ridge into an adjoining valley. Deep seepage can take place, but the rate of flow is determined by the in situ permeability.
3. Both the groundwater divide and piezometric level are at a lower elevation than the top water level but higher than that of the reservoir floor. In this case, the increase in groundwater head is low, and the flow from the reservoir may be initiated under conditions of low piezometric pressure in the reservoir flanks.
4. The water table is depressed below the base of the reservoir floor. This indicates deep drainage of the rock mass or very limited recharge. A depressed water table does not necessarily mean that reservoir construction is out of the question but groundwater recharge will take place on filling that will give rise to a changed hydrogeological environment as the water table rises. In such instances, the impermeability of the reservoir floor is important. When permeable beds are more or less saturated, particularly when they have no outlet, seepage is decreased appreciably. At the same time, the accumulation of silt on the floor of a reservoir tends to reduce seepage. If, however, the permeable beds contain large pore spaces or discontinuities and they drain from the reservoir, then seepage continues.

Troubles from seepage usually can be controlled by exclusion or drainage techniques. Cut-off trenches, carried into bedrock, may be constructed across cols occupied by permeable deposits. Grouting may be effective where localized fissuring is the cause of leakage.

Impervious linings consume large amounts of head near the source of water, thereby reducing hydraulic gradients and saturation at the points of exit and increasing resistance to seepage loss. Clay blankets or layers of silt have been used to seal exits from reservoirs.

Because of the occurrence of permeable contacts, close jointing, pipes, and the possible presence of tunnels and cavities, recent accumulations of basaltic lava flows can prove highly leaky rocks with respect to watertightness. Lava flows frequently are interbedded, often in an irregular fashion, with pyroclastic deposits. Deposits of ash and cinders tend to be highly permeable.

Reservoir sites in limestone terrains vary considerably in their suitability. Massive horizontally bedded limestones, relatively free from solution features, form excellent sites. On the other hand, well-jointed, cavernous and deformed limestones are likely to present problems in terms of stability and watertightness. Serious leakage usually has taken place as a result of cavernous conditions that are not fully revealed or appreciated at the site investigation stage. Indeed, sites are best abandoned where large numerous solution cavities extend to considerable depths. Where the problem is not so severe, solution cavities can be cleaned and grouted (Kannan, 2003). In addition, reference has been made by Milanovic (2003) to the application of a blanket of shotcrete to seal areas of karstic rock in reservoir basins. However, wet rock surfaces are not suitable as far as the application of shotcrete is concerned and neither is it wise to allow groundwater pressure to build up beneath shotcrete.

Sinkholes and caverns can develop in thick beds of gypsum more rapidly than they can in limestone. Indeed, in the United States, they have been known to form within a few years in areas where beds of gypsum are located below reservoirs. Extensive surface cracking and subsidence has occurred in Oklahoma and New Mexico due to the collapse of cavernous gypsum. The problem is accentuated by the fact that gypsum is weaker than limestone and, therefore, collapses more readily. Uplift is a problem that has been associated with the hydration of anhydrite beneath reservoirs.

Buried channels may be filled with coarse granular stream deposits or deposits of glacial origin and, if they occur near the perimeter of a reservoir, they almost invariably pose leakage problems. Indeed, leakage through buried channels, via the perimeter of a reservoir, is usually more significant than through the main valley. Hence, the bedrock profile, the type of deposits and groundwater conditions should be determined.

A thin layer of relatively impermeable superficial material does not necessarily provide an adequate seal against seepage. A controlling factor in such a situation is the groundwater pressure immediately below the blanket. Where artesian conditions exist, springs may break the thinner parts of the superficial cover. If the water table below such a blanket is depressed, then there is a risk that the weight of water in the reservoir may puncture it. What is more, on filling a reservoir, there is a possibility that the superficial material may be ruptured or partially removed to expose the underlying rocks. This happened at the Monte Jacques Reservoir in northern Spain where alluvial deposits covered cavernous limestone. The alluvium was washed away to expose a large sinkhole down which reservoir water escaped.

Leakage along faults generally is not a serious problem as far as reservoirs are concerned since the length of the flow path usually is too long. However, fault zones occupied by permeable fault breccia that extend from the reservoir to run beneath the dam must be given special consideration. When the reservoir basin is filled, the hydrostatic pressure may cause removal of loose material from such fault zones and thereby accentuate leakage. Permeable fault zones can be grouted, or if a metre or so wide, excavated and filled with rolled clay or concrete.

Stability of the Sides of Reservoirs

The formation of a reservoir upsets the groundwater regime and represents an obstruction to water flowing downhill. The greatest change involves the raising of the water table. Some soils or rocks, which formerly were not within the zone of saturation, may become unstable and fail, as saturated material is weaker than unsaturated. This can lead to slumping and sliding on the flanks of a reservoir (Riemer, 1995). In glaciated valleys, morainic material often rests on a rock slope smoothed by glacial erosion, which accentuates the problem of slip. Landslides that occur after a reservoir is filled reduce its capacity. Also, ancient landslipped areas that occur on the rims of a reservoir may present a leakage hazard and could be reactivated.

The worst man-induced landslide on record took place in the Vajont Reservoir in northern Italy in 1963 (Semenza. and Ghirotti, 2000). More than 300×10^6 m^3 moved downhill with such momentum that it crossed the 99 m wide gorge and rode 135 m up the opposite side. It filled the reservoir for a distance of 2 km with slide material, which in places reached heights of 175 m and displaced water in the reservoir, thereby generating huge waves that overtopped both abutments to a height of some 100 m above the crest of the dam.

Sedimentation in Reservoirs

Although it is seldom a decisive factor in determining location, sedimentation in reservoirs is an important problem in some countries (De Souza et al., 1998). For example, investigations in the United States suggest that sedimentation will limit the usefulness of most reservoirs to less than 200 years. Sedimentation in a reservoir may lead to one or more of its major functions being seriously curtailed or even to it becoming inoperative. For instance, Tate and Farquharson (2000) noted that the useful life of Tabela Reservoir on the River Indus, Pakistan, is threatened by a sediment delta that is approaching the intake tunnels of the dam. In a small reservoir, sedimentation may affect the available carry-over water supply seriously and ultimately necessitate abandonment.

In those areas where streams carry heavy sediment loads, the rates of sedimentation must be estimated accurately in order that the useful life of any proposed reservoir may be determined. The volume of sediment carried varies with stream flow, but usually the peak sediment load will occur prior to the peak stream flow discharge. Frequent sampling accordingly must be made to ascertain changes in sediment transport. Volumetric measurements of sediment in reservoirs are made by soundings taken to develop the configuration of the reservoir sides and bottom below the water surface.

Size of a drainage basin is the most important consideration as far as sediment yield is concerned, the rock types, drainage density and gradient of slope also being important. The sediment yield also is influenced by the amount and seasonal distribution of precipitation and the vegetative cover. Poor cultivation practices, overgrazing, improper disposal of mine waste and other human activities may accelerate erosion or contribute directly to stream loads.

The ability of a reservoir to trap and retain sediment is known as its trap efficiency and is expressed as the percentage of incoming sediment that is retained. Trap efficiency depends on total inflow, rate of flow, sediment characteristics and the size of the reservoir.

Dams and Dam Sites

The type and size of dam constructed depends on the need for and the amount of water available, the topography and geology of the site, and the construction materials that are readily obtainable. Dams can be divided into two major categories according to the type of material with which they are constructed, namely, concrete dams and earth dams. The former category can be subdivided into gravity, arch and buttress dams, whereas rolled fill and rockfill embankments comprise the other. As far as dam construction is concerned, safety must be the primary concern, this coming before cost. Safety requires that the foundations and abutments be adequate for the type of dam selected.

A gravity dam is a rigid monolithic structure that is usually straight in plan, although sometimes it may be slightly curved. Its cross section is roughly trapezoidal. Generally, gravity dams can tolerate only the smallest differential movements, and their resistance to dislocation by the hydrostatic pressure of the reservoir water is due to their own weight. A favourable site is usually one in a constricted area of a valley where sound bedrock is reasonably close to the surface, both in the floor and abutments.

An arch dam consists of a concrete wall, of high-strength concrete, curved in plan, with its convex face pointing upstream (Fig. 9.19). Arch dams are relatively thin walled and lighter in weight than gravity dams. They stand up to large deflections in the foundation rock, provided that the deflections are uniformly distributed. They transmit most of the horizontal thrust of the reservoir water to the abutments by arch action and this, together with their relative

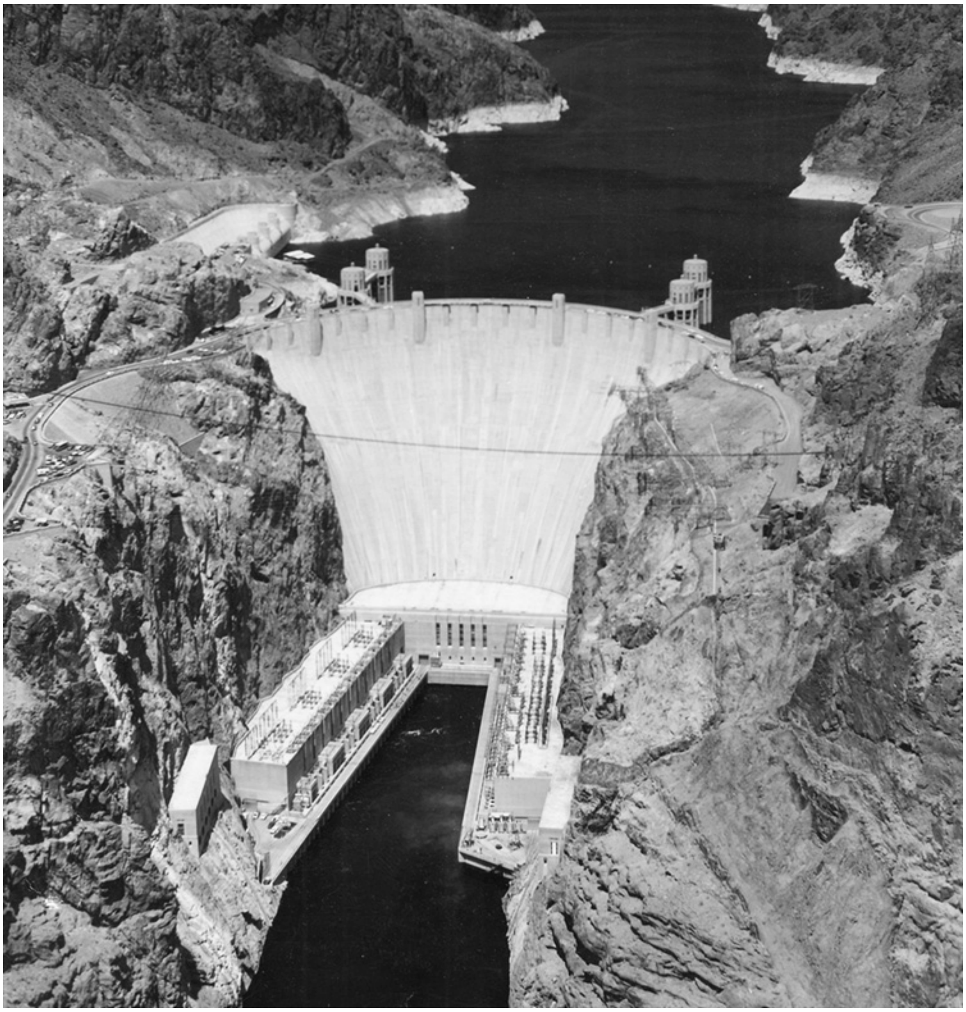


Figure 9.19

Hoover Dam, Colorado, completed in the 1930s but still one of the largest and most impressive arch dams in the world.

thinness, means that they impose high stresses on narrow zones at the base, as well as the abutments of the dam. Therefore, the strength of the rock mass at the abutments, and below and immediately down-valley of the dam must be unquestionable, and the modulus of elasticity must be high enough to ensure that its deformation under thrust from the arch is not so great as to induce excessive stresses in the arch. Ideal locations for arch dams are provided by narrow gorges where the walls are capable of withstanding the thrust produced by the arch action.

Buttress dams provide an alternative to other concrete dams in locations where the foundation rocks are competent. A buttress dam consists principally of a slab of reinforced concrete that slopes upstream and is supported by a number of buttresses whose axes are normal to the slab (Fig. 9.20). The buttresses support the slab and transmit the water load to the foundation. They are rather narrow and act as heavily loaded walls, thus exerting substantial unit pressures on the foundations.

Earth dams are embankments of earth with an impermeable core to control seepage (Fig. 9.21). This usually consists of clayey material. If sufficient quantities are not available, then concrete or asphaltic concrete membranes are used. The core normally is extended as a cut-off or grout curtain below ground level when seepage beneath the dam has to be controlled.

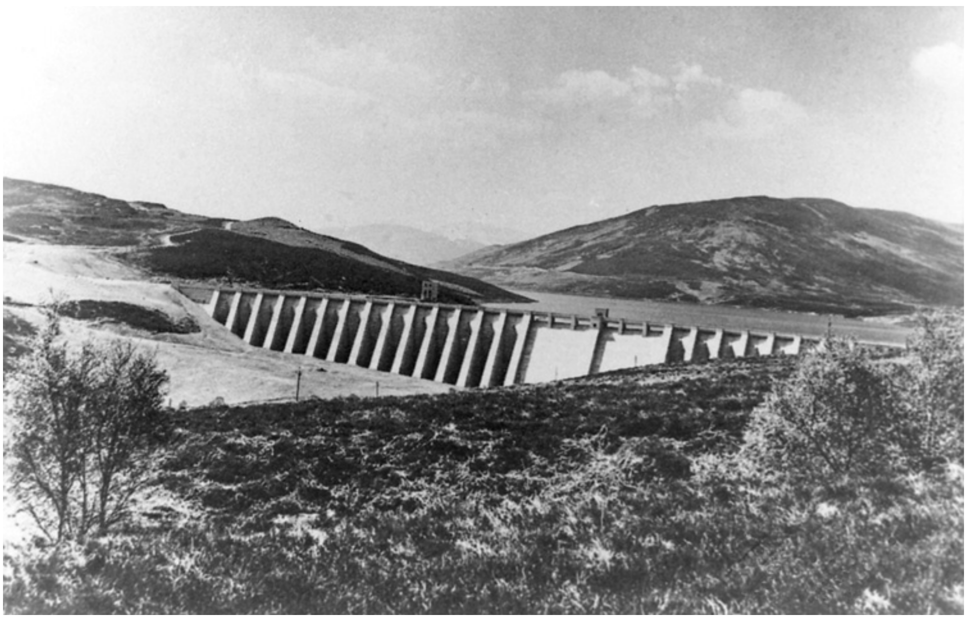


Figure 9.20

Errochty Dam, Scotland, an example of a buttress dam.

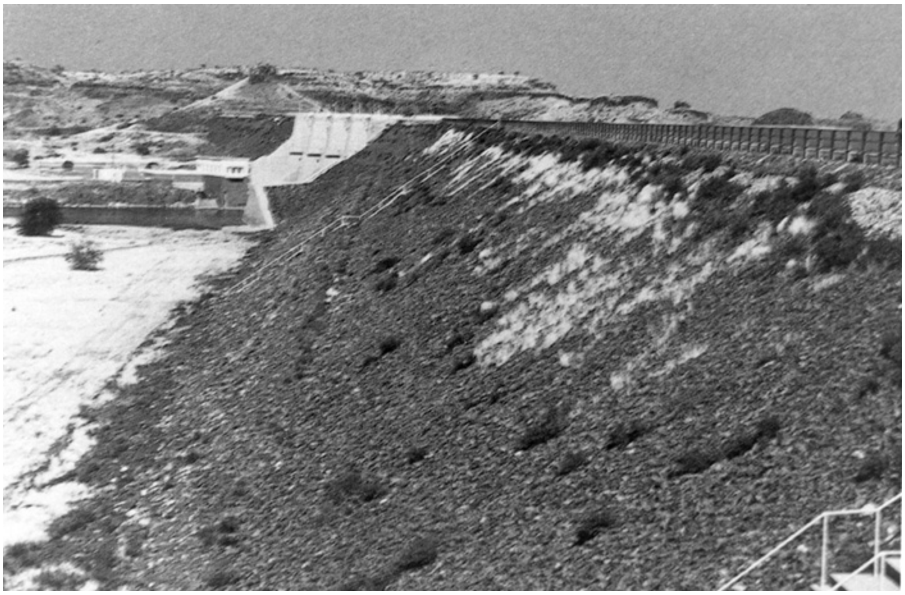


Figure 9.21

Harddap Dam, near Mariental, Namibia, and example of an embankment dam.

Drains of sand and/or gravel installed beneath or within the dam also afford seepage control. Because of their broad base, earth dams impose much lower stresses on the foundation materials than concrete dams. Furthermore, they can accommodate deformation such as that due to settlement more readily. As a consequence, earth dams have been constructed on a great variety of foundations ranging from weak unconsolidated stream or glacial deposits to high-strength rocks.

An earth dam may be zoned or homogeneous, the former type being more common. A zoned dam is a rolled fill dam composed of several zones that increase in permeability from the core towards the outer slopes. The number of zones depends on the type and amount of borrow material available. Stability of a zoned dam is due mostly to the weight of the heavy outer zones.

If there is only one type of borrow material readily available, a homogeneous embankment is constructed. In other words, homogeneous dams are constructed entirely or almost entirely of one type of material. The latter is usually fine-grained, although sand and sand–gravel mixtures have been used.

Rockfill dams usually consist of three basic elements — a loose rockfill dump, which forms the bulk of the dam and resists the thrust of the reservoir; an impermeable facing or membrane on the upstream side or an impermeable core; and rubble masonry in between to act as a cushion for the membrane and to resist destructive deflections. Consolidation of the main rock body may leave the face unsupported with the result that cracks form through which seepage can occur. Flexible asphalt membranes overcome this problem.

Some sites that are geologically unsuitable for a specific type of dam design may support one of composite design. For example, a broad valley that has strong rocks on one side and weaker ones on the other possibly can be spanned by a combined gravity and embankment dam, that is, a composite dam (Fig. 9.22).

The construction of a dam and the filling of a reservoir behind it impose a load on the sides and floor of a valley, creating new stress conditions. These stresses must be analyzed so that there is ample assurance that there will be no possibility of failure. A concrete dam behaves as a rigid monolithic structure, the stress acting on the foundation being a function of the

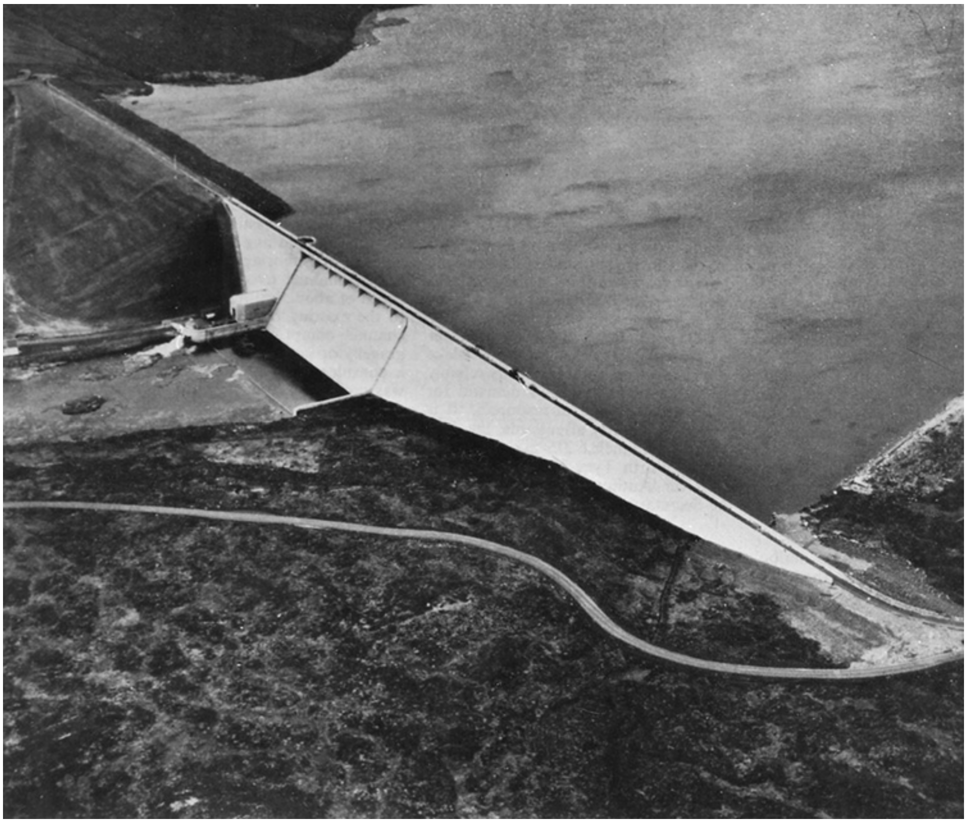


Figure 9.22

Cow Green Dam in Teesdale, northeast England, an example of a composite dam.

weight of the dam as distributed over the total area of the foundation. In contrast, an earthfill dam exhibits semi-plastic behaviour, and the pressure on the foundation at any point depends on the thickness of the dam above that point. Vertical static forces act downward and include both the weight of the structure and the water, although a large part of the dam is submerged and, therefore, the buoyancy effect reduces the influence of the load. The most important dynamic forces acting on a dam are wave action, overflow of water and seismic shocks.

Horizontal forces are exerted on a dam by the lateral pressure of water behind it. These, if excessive, may cause concrete dams to slide. The tendency towards sliding at the base of such dams is of particular significance in fissile rocks such as shales, slates and phyllites. Weak zones, such as interbedded ashes in a sequence of basalt lava flows, can prove troublesome. The presence of flat-lying joints may destroy much of the inherent shear strength of a rock mass and reduce the problem of resistance of a foundation to horizontal forces to one of sliding friction, so that the roughness of joint surfaces becomes a critical factor. The rock surface should be roughened to prevent sliding, and keying the dam some distance into the foundation is advisable. Another method of reducing sliding is to give a downward slope to the base of the dam in the upstream direction of the valley.

Variations in pore water pressure cause changes in the state of stress in rock masses. They reduce the compressive strength of rocks and cause an increase in the amount of deformation they undergo. Pore water also may be responsible for swelling in certain rocks and for acceleration in their rate of alteration. Pore water in the stratified rocks of a dam foundation reduces the coefficient of friction between the individual beds, and between the foundation and the dam.

Percolation of water through the foundations of concrete dams, even when the rock masses concerned are of good quality and low permeability, is a decisive factor in the safety and performance of such dams. Such percolation can remove filler material that may be occupying joints that, in turn, can lead to differential settlement of the foundations. It also may open joints, which decreases the strength of the rock mass.

In highly permeable rock masses, excessive seepage beneath a dam may damage the foundation. Seepage rates can be lowered by reducing the hydraulic gradient beneath the dam by incorporating a cut-off into the design. A cut-off lengthens the flow path, reducing the hydraulic gradient. It extends to an impermeable horizon or some specified depth and usually is located below the upstream face of the dam. The rate of seepage also can be effectively reduced by placing an impervious earthfill against the lower part of the upstream face of a dam.

Uplift pressure acts against the base of a dam and is caused by water seeping beneath it that is under hydrostatic head from the reservoir. Uplift pressure should be distinguished from the pore water pressure in the material beneath a dam. The uplift pressure on the heel of a dam is equal to the depth of the foundation below water level multiplied by the unit weight of the water. In the simplest case, it is assumed that the difference in hydraulic heads between the heel and the toe of the dam is dissipated uniformly between them. The uplift pressure can be reduced by allowing water to be conducted downstream by drains incorporated into the foundation and base of the dam.

When load is removed from a rock mass on excavation, it is subject to rebound. The amount of rebound depends on the modulus of elasticity of the rocks concerned, the larger the modulus of elasticity, the smaller the rebound. The rebound process in rocks generally takes considerable time to achieve completion and will continue after a dam has been constructed if the rebound pressure or heave developed by the foundation material exceeds the effective weight of the dam. Hence, if heave is to be counteracted, a dam should impose a load on the foundation equal to or slightly in excess of the load removed.

All foundation and abutment rocks yield elastically to some degree. In particular, the modulus of elasticity of a rock mass is of primary importance as far as the distribution of stresses at the base of a concrete dam is concerned. What is more, tensile stresses may develop in concrete dams when the foundations undergo significant deformation. The modulus of elasticity is used in the design of gravity dams for comparing the different types of foundation rocks with each other and with the concrete of the dam. In the design of arch dams, if Young's modulus of the foundation has a lower value than that of the concrete or varies widely in the rocks against which the dam abuts, then dangerous stress conditions may develop in the dam. The elastic properties of a rock mass and existing strain conditions assume importance in proportion to the height of a dam since this influences the magnitude of the stresses imparted to the foundation and abutments. The influence of geological structures in lowering Young's modulus must be accounted for by the provision of adequate safety factors. It should also be borne in mind that blasting during excavation of foundations can open up fissures and joints that leads to greater deformability of the rock mass. The deformability of the rock mass, any possible settlements and the amount of increase of deformation with time can be taken into consideration by assuming lower moduli of elasticity in the foundation or by making provisions for prestressing.

Geology and Dam Sites

Of the various natural factors that directly influence the design of dams, none is more important than the geological, not only do they control the character of the foundation but they also govern the materials available for construction. The major questions that need answering include the depth at which adequate foundations exist, the strengths of the rock masses involved, the likelihood of water loss and any special features that have a bearing on excavation. The character of the foundations upon which dams are built and their reaction to the new conditions of stress and strain, of hydrostatic pressure and of exposure to weathering must be ascertained so that the proper factors of safety may be adopted to ensure against subsequent failure. Excluding the weaker types of compaction shales, mudstones, marls, pyrolasts and certain very friable types of sandstone, there are few foundation materials deserving the name rock that are incapable of resisting the bearing loads even of high dams.

In their unaltered state, plutonic igneous rocks essentially are sound and durable, with adequate strength for any engineering requirement. In some instances, however, intrusives may be highly altered by weathering or hydrothermal attack. Generally, the weathered product of plutonic rocks has a large clay content, although that of granitic rocks is sometimes porous with a permeability comparable to that of medium-grained sand, so that it requires some type of cut-off or special treatment of the upstream surface.

Thick massive basalts make satisfactory dam sites but many basalts of comparatively young geological age are highly permeable, transmitting water via their open joints, pipes, cavities, tunnels, and contact zones. Foundation problems in young volcanic sequences are twofold. Firstly, weak beds of ash and tuff may occur between the basalt flows that give rise to problems of differential settlement or sliding. Secondly, weathering during periods of volcanic inactivity may have produced fossil soils, these being of much lower strength.

Rhyolites, and frequently andesites, do not present the same severe leakage problems as young basalt sequences. They frequently offer good foundations for concrete dams, although at some sites chemical weathering may mean that embankment designs have to be adopted.

Pyroclastics usually give rise to extremely variable foundation conditions due to wide variations in strength, durability and permeability. Their behaviour very much depends on their degree of induration, for example, many agglomerates have high enough strengths to support concrete dams and also have low permeabilities. By contrast, ashes are weak and often highly permeable. One particular hazard concerns ash not previously wetted, that is, it may be metastable and so undergoes a significant reduction in its void ratio on saturation. Clay/cement grouting at high pressures may turn ash into a satisfactory foundation. Ashes frequently are prone to sliding. Montmorillonite is not an uncommon constituent in these rocks when they are weathered, so that they may swell on wetting.

Fresh metamorphosed rocks such as quartzite and hornfels are very strong and afford excellent dam sites. Marble has the same advantages and disadvantages as other carbonate rocks. Generally, gneiss has proved a good foundation rock for dams.

Cleavage, schistosity and, to a lesser extent, foliation in regional metamorphic rocks may adversely affect their strength and make them more susceptible to decay. Moreover areas of regional metamorphism usually have suffered extensive folding so that rocks may be fractured and deformed. Some schists, slates and phyllites are variable in quality, some being excellent for dam site purposes, others, regardless of the degree of their deformation or weathering, are so poor as to be wholly undesirable in foundations and abutments. For instance, talc, chlorite and sericite schists are weak rocks containing closely spaced planes of schistosity.

Some schists become slippery upon weathering and, therefore, fail under moderately light loads. On the other hand, slates and phyllites tend to be durable. Although slates and phyllites are suitable for concrete dams where good load-bearing strata occur at a relatively shallow depth, problems may arise in excavating broad foundations. Particular care is required in blasting slates, phyllites and schists, otherwise considerable overbreak or shattering may result. It may be advantageous to use smooth blasting for final trimming purposes. When compacted in lifts using a vibratory roller, these rocks tend to break down to give a well-graded permeable fill. Consequently, rock fill embankments are being increasingly adopted at such sites.

Joints and shear zones are responsible for the unsound rock encountered at dam sites on plutonic and metamorphic rocks. Unless they are sealed, they may permit leakage through foundations and abutments. Slight opening of joints on excavation leads to imperceptible rotations and sliding of rock blocks, large enough to appreciably reduce the strength and stiffness of the rock mass. Sheet or flat-lying joints tend to be approximately parallel to the topographic surface and introduce a dangerous element of weakness into valley slopes. Their width varies and, if they remain untreated, large quantities of water may escape through them from the reservoir. Indeed, Terzaghi (1962) observed that the most objectionable feature in terms of the foundation at Mammoth Pool Dam, California, which is in granodiorite, was the sheet joints orientated parallel to the rock surface. Moreover, joints may transmit hydrostatic pressures into the rock masses downstream from the abutments that are high enough to dislodge sheets of rock. If a joint is very wide and located close to the rock surface, it may close up under the weight or lateral pressure exerted by the dam and cause differential settlement.

Sandstones have a wide range of strength, depending largely on the amount and type of cement-matrix material occupying the voids. With the exception of shaley sandstone, sandstone is not subject to rapid surface deterioration on exposure. As a foundation rock, even poorly cemented sandstone is not susceptible to plastic deformation. However, friable sandstones introduce problems of scour at the foundation. Moreover, sandstones are highly vulnerable to the scouring and plucking action of the overflow from dams and have to be adequately protected by suitable hydraulic structures. A major problem of dam sites located in sandstones results from the fact that they normally are transected by joints, which reduce resistance to sliding. Generally, however, sandstones have high coefficients of internal friction that give them high shearing strengths, when restrained under load.

Sandstones frequently are interbedded with shale. These layers of shale may constitute potential sliding surfaces. Sometimes, such interbedding accentuates the undesirable properties of the shale by permitting access of water to the shale–sandstone contacts. Contact seepage may weaken shale surfaces and cause sliding in formations that dip away from abutments and spillway cuts. Severe uplift pressures also may develop beneath beds of shale in a dam foundation and appreciably reduce its resistance to sliding. Foundations and abutments composed of interbedded sandstones and shales also present problems of settlement and rebound, the magnitude of these factors depending on the character of the shales.

The permeability of sandstone depends on the amount of cement in the voids and, more particularly, on the incidence of discontinuities. The porosity of sandstones generally does not introduce leakage problems of moment, though there are exceptions. The sandstones in a valley floor may contain many open joints that wedge out with depth, and these often are caused by rebound of interbedded shales. Conditions of this kind in the abutments and foundations of dams greatly increase the construction costs for several reasons. They have a marked influence on the depth of stripping, especially in the abutments. They must be cut off by pressure grouting and drainage, for the combined purposes of preventing excessive leakage and reducing the undesirable uplift effects of the hydrostatic pressure of reservoir water on the base of the dam or on the base of some bedding contact within the dam foundation.

Limestone dam sites vary widely in their suitability. Thick-bedded, horizontally lying limestones, relatively free from solution cavities, afford excellent dam sites. Also, limestone requires no special treatment to ensure a good bond with concrete. On the other hand, thin-bedded, highly folded or cavernous limestones are likely to present serious foundation or abutment problems involving bearing capacity or watertightness or both (Soderburg, 1979). Resistance to sliding involves the shearing strength of limestone. If the rock mass is thin bedded, a possibility of sliding may exist. This should be guarded against by suitably keying the dam structure into the foundation rock. Beds separated by layers of clay or shale, especially those inclined downstream, may serve as sliding planes under certain conditions.

Some solution features are always be present in limestone. The size, form, abundance and downward extent of these features depend on geological structure and presence of interbedded impervious layers. Individual cavities may be open, they may be partially or completely filled with clay, silt, sand or gravel mixtures or they may be water-filled conduits. Solution cavities present numerous problems in the construction of large dams, among which bearing capacity and watertightness are paramount. Few dam sites are so bad that it is impossible to construct safe and successful structures upon them but the cost of the necessary remedial treatment may be prohibitive. In fact, dam sites should be abandoned where the cavities are large and numerous, extending to considerable depths. Sufficient bearing strength generally may be obtained in cavernous rock by deeper excavation than otherwise would be necessary. Watertightness may be attained by removing the material from cavities, and refilling with concrete. Small filled cavities may be sealed effectively by washing out and then grouting with cement. The establishment of a watertight cut-off through cavernous limestone presents difficulties in proportion to the size and extent of the solution openings. Grouting has not always proved successful in preventing water loss from reservoirs on karstic terrains. For example, Bozovic et al. (1981) referred to large caverns in limestone at the Keban Dam site in Turkey that exceeded 100,000 m^3 in volume. In fact, despite 36,000 m of exploratory drilling and 11 km of exploratory adits, a huge cavern over 600,000 m^3 went undiscovered. This illustrates the fact that risk in karstic areas cannot be completely eliminated even by intensive site investigation. Even though these caverns were filled with large blocks of rock ($0.5 \times 0.5 \times 0.5$ m) and aggregate, and an extensive grouting programme carried out, leakage on reservoir impoundment amounted to some 26 $m^3\ s^{-1}$. A classic case of leakage was associated with the Hales Bar Dam, Tennessee, which was founded on the Bangor Limestone. After completion of the dam in 1917, it underwent several episodes of extensive grout treatment. None were successful, and leakage had increased to more than 54 $m^3\ s^{-1}$ by the late 1950s. Consequently, the dam was demolished in 1968. Another difficult project has been described by Turkmen et al. (2002), namely, the Kalecik Dam in Turkey. There seepage through the karstic limestone beneath led to a 200 m long and 60 m deep grout curtain being constructed beneath the axis of this rockfill dam. Unfortunately, this did not solve the seepage problem. A further investigation showed that seepage paths existed between the dam and the spillway. Therefore, it was recommended that a new grout curtain be constructed beneath the spillway.

The removal of evaporites by solution can result in subsidence and collapse of overlying strata. Indeed, cavities have been known to form in the United States within a matter of a few years where thick beds of gypsum occurred beneath dams. Brune (1965) reported extensive surface cracking and subsidence in reservoir areas in Oklahoma and New Mexico due to the collapse of cavernous gypsum. He also noted that a sinkhole appeared in the sediment pool shortly after the completion of the Cavalry Creek Dam, Oklahoma, which caused much water to be lost. Investigations, however, have shown that when anhydrite and gypsum are interbedded with marl (mudstone), they generally are sound.

Well cemented shales, under structurally sound conditions, present few problems at dam sites, though their strength limitations and elastic properties may be factors of importance in the design of concrete dams of appreciable height. They, however, have lower moduli of elasticity and lower shear strength values than concrete and, therefore, are unsatisfactory foundation materials for arch dams. Moreover, if the lamination is horizontal and well developed, then the foundations may offer little shear resistance to the horizontal forces exerted by a dam. A structure keying the dam into such a foundation is then required.

Severe settlements may take place in low grade compaction shales. As a consequence, such sites are generally developed with earth dams, but associated concrete structures such as spillways will involve these problems. Rebound in deep spillway cuts may cause buckling of spillway linings, and differential rebound movements in the foundations may require special design provisions.

The stability of slopes in cuts is one of the major problems in shale both during and after construction. Cuttings in shale above structures must be made stable. This problem becomes particularly acute in dipping formations and in formations containing montmorillonite.

Earth dams are usually constructed on clay soils as they have insufficient load-bearing properties required to support concrete dams. Beneath valley floors, clays may be contorted, fractured and softened due to valley bulging so that the load of an earth dam may have to spread over wider areas than is the case with shales and mudstones. Settlement beneath an embankment dam constructed on soft clay soils can present problems and may lead to the development of excess pore water pressures in the foundation soils (Olson, 1998). Rigid ancillary structures necessitate spread footings or raft foundations. Deep cuts involve problems of rebound if the weight of removed material exceeds that of the structure. Slope stability problems also arise, with rotational slides being a hazard.

Among the many manifestations of glaciation are the presence of buried channels; disrupted drainage systems; deeply filled valleys; sand–gravel terraces; narrow overflow channels connecting open valleys; and extensive deposits of lacustrine silts and clays, till, and outwash sands and gravels. Deposits of peat and head (solifluction debris) may be interbedded with these glacial deposits. Consequently, some glacial deposits may be notoriously variable in composition, both laterally and vertically. As a result, some dam sites in glaciated areas are among the most difficult to appraise on the basis of surface evidence. Knowledge of the preglacial, glacial and postglacial history of a locality is of importance in the search for the most practical sites. A primary consideration in glacial terrains is the discovery of sites where rock foundations are available for spillway, outlet and powerhouse structures. Generally, earth dams are constructed in areas of glacial deposits. Concrete dams, however, are feasible in post-glacial, rock-cut valleys, and composite dams are practical in valleys containing rock benches.

The major problems associated with foundations on alluvial deposits generally result from the fact that the deposits are poorly consolidated. Silts and clays are subject to plastic deformation or shear failure under relatively light loads and undergo consolidation for long periods of time when subjected to appreciable loads. Embankment dams are normally constructed on such soils as they lack the load-bearing capacity necessary to support concrete dams. The slopes of an embankment dam may be flattened in order to mobilize greater foundation shear strength, or berms may be introduced into the slope. Nonetheless, many large embankment dams have been built on such materials, but this demands a thorough exploration and testing programme in order to design safe structures. Soft alluvial clays at ground level generally have been removed if economically feasible. Where soft alluvial clays are not more than 2.3 m thick, they may consolidate during construction if covered with a drainage blanket, especially if they are resting on sand and gravel. It may be necessary in thicker deposits to incorporate vertical drains within the clays (Almeida et al., 2000). On the other hand, coarser sands and gravels undergo comparatively little consolidation under load and therefore often afford good foundations for earth dams. Their primary problems result from their permeability. Alluvial sands and gravels form natural drainage blankets under an earth or rock fill dam, so that seepage through them beneath the dam must be cut off. Problems relating to underseepage through pervious strata may be tackled by a cut-off trench, if the depth to bedrock is not too great or by a grout curtain. Otherwise, underseepage may be checked by the construction of an impervious upstream blanket to lengthen the path of percolation and the installation on the downstream side of suitable drainage facilities to collect the seepage.

Talus or scree may clothe the lower slopes in mountainous areas and, because of its high permeability, must be avoided in the location of a dam site, unless it is sufficiently shallow to be economically removed from under the footprint of the dam.

Landslips are a common feature of valleys in mountainous areas, and large slips often cause narrowing of a valley that therefore looks topographically suitable for a dam. Unless they are shallow seated and can be removed or effectively drained, it is prudent to avoid landslipped areas in dam location, because their unstable nature may result in movement during construction or, subsequently, on filling or drawdown of the reservoir.

Fault zones may be occupied by shattered or crushed material and so represent zones of weakness that may give rise to landslip upon excavation for a dam. The occurrence of faults in a river is not unusual, and this generally means that the material along the fault zone is highly altered. A deep cut-off is necessary in such a situation.

In most known instances of historic fault breaks, the fracturing has occurred along a pre-existing fault. Fault movement not only occurs in association with large and infrequent earthquakes but it also occurs in association with small shocks and continuous slippage known as fault creep. Earthquakes resulting from displacement and energy release on one fault can sometimes trigger small displacements on other unrelated faults many kilometres distant. Breaks on subsidiary faults have occurred at distances as great as 25 km from the main fault, obviously with increasing distance from the main fault, the amount of displacement decreases.

Individual breaks along faults during earthquakes have ranged in length from less than a kilometre to several hundred kilometres. However, the length of the fault break during a particular earthquake is generally only a fraction of the true length of the fault. The longer fault breaks have greater displacements and generate larger earthquakes. The maximum displacement is less than 6 m for the great majority of fault breaks, and the average displacement along the length of the fault is less than half the maximum. These figures suggest that zoned embankment dams can be safely built at sites with active faults.

All major faults located in regions where strong earthquakes have occurred should be regarded as potentially active unless convincing evidence exists to the contrary (Sherard et al., 1974). In stable areas of the world, little evidence exists of notable fault displacements in the recent past. Nevertheless, an investigation should be carried out to confirm the absence of active faults at or near any proposed major dam in any part of the world.

Construction Materials for Earth Dams

Wherever possible, construction materials for an earth dam should be obtained from within the future reservoir basin. Accordingly, the investigation for the dam site and the surrounding area should determine the availability of impervious and pervious materials for the embankment, sand and gravel for drains and filter blankets, and stone for riprap.

In some cases, only one type of soil is readily obtainable for an earth dam. If this is impervious, then the design will consist of a homogeneous embankment, which incorporates a small amount of permeable material in order to control internal seepage. On the other hand, where sand and gravel are in plentiful supply, a very thin earth core may be built into the dam if enough impervious soil is available, otherwise an impervious membrane may be constructed of concrete or interlocking steel sheet piles. However, since concrete can withstand very little settlement, such core walls should be located on sound foundations.

Sites that provide a variety of soils lend themselves to the construction of zoned dams. The finer, more impervious materials are used to construct the core, whereas the coarser materials provide strength in the upstream and downstream zones.

Embankment soils need to develop high shear strength, low permeability and low water absorption, and undergo minimal settlement. This is achieved by compaction. The degree of compaction achieved is reflected by the dry density of the soil. The relationship between dry density and moisture content of a soil for a particular compactive effort is assessed by a compaction test.

Serious piping damage and failures have occurred when dispersive soils have been used for the construction of earth dams. Early indications of piping take the form of small leakages of muddy water from an earth embankment after initial filling of the reservoir. Dispersive erosion may be caused by initial seepage through the more pervious areas in an earth dam. This is especially the case in areas in which compaction may not be so effective, such as at the contacts with conduits, against concrete structures and the foundation interface; through desiccation cracks; or through cracks formed by differential settlement or hydraulic fracturing. In fact, most earth dams that have failed in South Africa did so on first wetting because that is when the fill is most vulnerable to hydraulic fracturing (Bell and Maud, 1994). Fractures represent paths along which piping can develop. The pipes can enlarge rapidly, and this can lead to failure of a dam. Far more failures have occurred in small homogeneous earth dams, which generally are more poorly engineered and seldom have filters, than in major dams (see Fig. 5.9).

River Diversion

Wherever dams are built, there are problems concerned with keeping the associated river under control. These have a greater influence on the design of an embankment dam than a concrete dam. In narrow, steep-sided valleys, the river is diverted through a tunnel or conduit before the foundation is completed over the floor of the river. However, the abutment sections of an embankment dam can be constructed in wider valleys prior to river diversion. In such instances, suitable borrow materials must be set aside for the closure section, as this often has to be constructed rapidly so that overtopping is avoided. But rapid placement of the closure section can give rise to differential settlement and associated cracking. Hence, extra filter drains may be required to control leakage through such cracks. Compaction of the closure section at a higher average water content means that it can adjust more easily to differential settlement without cracking.

Ground Improvement

Grouting has proved effective in reducing percolation of water through dam foundations, and its introduction into dam construction has allowed considerable cost saving by avoiding the use of deep cut-off and wing trenches. Consequently, many sites that previously were considered unsuitable because of adverse geological conditions can now be used.

Initial estimates of the groutability of ground frequently been have based on the results of pumping-in tests, in which water is pumped into the ground via a drillhole. Lugeon (1933) suggested that grouting beneath concrete gravity dams was necessary when the permeability exceeded 1 lugeon unit (i.e. a flow of 1 l m^{-1} min^{-1} at a pressure of 1 MPa). However, this standard has been relaxed in modern practice, particularly for earth dams and for foundations in which seepage is acceptable in terms of lost storage and non-erodibility of foundation or core materials (Houlsby, 1990).

The effect of a grout curtain is to form a wall of low permeability within the ground below a dam. Holes are drilled and grouted, from the base of the cut-off or heel trench downwards. Where joints are vertical, it is advisable to drill groutholes at a rake of 10–15°, since these cut across the joints at different levels, whereas vertical holes may miss them.

The rate at which grout can be injected into the ground generally increases with an increase in the grouting pressure, but this is limited since excessive pressures cause the ground to fracture and lift (Kennedy, 2001). The safe maximum pressure depends on the weight of overburden, the strength of the ground, the in situ stresses and the pore water pressures. However, there is no simple relationship between these factors and safe maximum grouting pressure. Hydraulic fracture tests may be used, especially in fissile rocks, to determine the most suitable pressures or the pressures may be related to the weight of overburden.

Once the standard of permeability has been decided, for the whole or a section of a grout curtain, it is achieved by split spacing or closure methods in which primary, secondary, tertiary, etc., sequences of grouting are carried out until water tests in the groutholes approach the required standard (Houlsby, 1990). In multiple-row curtains, the outer rows should be completed first, thereby allowing the inner rows to effect closure on the outer rows. A spacing of 1.5 m between rows usually is satisfactory. The upstream row should be the tightest row, with the tightness decreasing downstream. Single-row curtains generally are constructed by drilling alternate holes first and then completing the treatment by intermediate holes. Ideally, a grout curtain is taken to a depth where the requisite degree of tightness is available naturally. This is determined either by investigation holes sunk prior to the design of the grout curtain, or by primary holes sunk during grouting (Ewert, 2005). The search usually does not go beyond a depth equal to the height of the storage head above ground surface.

Consolidation grouting is usually shallow, the holes seldom extending more than 10 m. It is intended to improve jointed rock by increasing its strength and reducing its permeability. Consolidation grouting in the foundation area increases the bearing capacity and minimizes settlement. The extent of consolidation grouting upstream and downstream of the grout curtain depends on the conditions in the upper zone of the foundation (Kutzner, 1996). Consolidation grouting also improves the contact between concrete and rock, and makes good any slight loosening of the rock surface due to blasting operations. In addition, it affords a degree of homogeneity to the foundation that is desirable if differential settlement and unbalanced stresses are to be avoided. In other words, the grout increases rock stiffness and attempts to bring Young's modulus to the required high uniform values. Holes usually are drilled normal to the foundation surface but they may be orientated to intersect specific features in certain instances. They are set out on a grid pattern at 3 to 14 m centres, depending on the nature of the rock. Consolidation grouting must be completed before the construction of a dam begins.

Casagrande (1961) cast doubts on the need for grout curtains, maintaining that a single-row grout curtain constructed prior to reservoir filling frequently is inadequate. What is more, he stated that grouting was useless as far as reducing water pressures was concerned and that drainage systems were the only efficient method of controlling the piezometric level and therefore uplift forces along a dam foundation. He further maintained that drainage is the only efficient treatment available for rock of low hydraulic conductivity that contains fine fissures. Drainage can control the hydraulic potential on the downstream side of a dam, thus achieving what is required of a grout curtain except, of course, that drainage does not reduce the amount of leakage. However, leakage is not of consequence in rock masses where the hydraulic conductivity is low. In other words, Casagrande contended that for fissured rocks of low permeability (i.e. less than 5 lugeon units), drainage generally is essential, whereas grouting constitutes a wasted effort. Conversely, if the permeability is high (in excess of 50 lugeon units), grouting is necessary to control groundwater leakage beneath a dam.

Highways

The location of highways and other routeways is influenced in the first instance by topography. Embankments, cuttings, tunnels and bridges (viaducts) can be constructed to carry roads and railroads with acceptable gradients through areas of more difficult terrain. Obviously, the construction of such structures increases the difficulty, time and cost of building routeways. Nonetheless, the distance between the centres that routeways connect has to be considered. Although geological conditions often do not determine the exact location of routeways, they can have a highly significant influence on their construction.

As highways are linear structures, they often traverse a wide variety of ground conditions along their length. In addition, the construction of a highway requires the excavation of soils and rocks, and stable foundations for the highway, as well as construction materials. The ground beneath roads and, more particularly, embankments, must have sufficient bearing capacity to prevent foundation failure and also be capable of preventing excess settlements due to the imposed load (Kezdi and Rethati, 1988). Very weak and compressible ground may need to be entirely removed before construction takes place, although this will depend on the quantity of material involved. For instance, if peat is less than 3–4 m thick and is underlain by a soil with a satisfactory bearing capacity, such as gravel or dense sand, then the peat may be removed prior to the construction of a road or, more particularly, an embankment (Perry et al., 2000). In some cases, heave that occurs due to the removal of load may cause problems. In other cases, improvement of the ground by the use of lime or cement stabilization, compaction, surcharging, the use of drainage, the installation of piles, stone columns or mattresses may need to be carried out prior to road and embankment construction (Cooper and Rose, 1999). Usually, the steepest side slopes possible are used when constructing cuttings and embankments, as this minimizes the amount of land required for the highway and the quantity of material that has to be moved. Obviously, attention must be given to the stability of slopes (Green and Hawkins, 2005). Slight variations in strength, spacing of discontinuities or the grade of weathering of rock masses can have an effect on the rate of excavation. Where the materials excavated are unsuitable for construction, considerable extra expense is entailed in disposing of waste and importing fill. Geological features such as faults, crush zones and solution cavities, as well as man-made features such as abandoned mine workings can cause difficulties during construction.

Normally, a road consists of a number of layers, each of which has a particular function. In addition, the type of pavement structure depends on the nature and number of the vehicles it has to carry, their wheel loads and the period of time over which it has to last (Brown, 1996). The wearing surface of a modern road consists either of “black-top” (i.e. bituminous bound aggregate) or a concrete slab, although a bituminous surfacing may overlie a concrete base. A concrete slab distributes the load that the road has to carry, whereas in a bituminous road, the load primarily is distributed by the base beneath. The base and sub-base below the wearing surface generally consist of granular material, although in heavy-duty roads, the base may be treated with cement. The subgrade refers to the soil immediately beneath the sub-base. However much the load is distributed by the layers above, the subgrade has to carry the load of the road structure plus that of the traffic. Consequently, the top of the subgrade may have to be strengthened by compaction or stabilization. The strength of the subgrade, however, does not remain the same throughout its life. Changes in its strength are brought about by changes in its moisture content, by repeated wheel loading, and in some parts of the world by frost action. Although the soil in the subgrade exists above the water table and beneath a sealed surface, this does not stop the ingress of water. As a consequence, partially saturated or saturated conditions can exist in the soil. Also, road pavements are constructed at a level where the subgrade is affected by wetting and drying, which may lead to swelling and shrinkage, respectively, if the subgrade consists of expansive clay. Such volume changes are non-uniform, and the associated movements may damage the pavement (Xeidakis et al., 2004). Irrecoverable plastic and viscous strains can accumulate under repeated wheel loading. In a bituminous pavement, repeated wheel loading can lead to fatigue and cracking, and rutting occurs as a result of the accumulation of vertical permanent strains.

Topographic and geological maps, remote sensing imagery and aerial photographs are used in highway location. These allow the preliminary plans and profiles of highways to be prepared. Geomorphological mapping has proved especially useful is relation to road construction in mountainous areas (Hearn, 2002). Geomorphological mapping helps to identify the general characteristics of an area in which a route is to be located. Moreover, it provides information on land-forming processes and geohazards that can affect road construction, on the character of natural slopes and on the location of construction materials, in addition to providing a basis on which to plan the subsequent site exploration. Such mapping can help the preliminary design of cut and fill slopes and land drainage, and help determine the approximate land-take requirements of a road. The site investigation provides the engineer with information on the ground and groundwater conditions on which a rational and economic design for a highway can be made. This information should indicate the suitability of the proposed location; the quantity of earthworks involved; the subsoil and surface drainage requirements; and the availability of suitable construction materials. Other factors that have to be taken account include the safe gradients for cuttings and embankments, locations of river crossings and possible ground treatment.

Unfortunately, many soils can prove problematic in highway engineering, because they expand and shrink, collapse, disperse, undergo excessive settlement, have a distinct lack of strength or are corrosive. Such characteristics may be attributable to their composition, the nature of their pore fluids, their mineralogy or their fabric. Frost heave can cause serious damage to roads, leading to their break-up. Furthermore, the soil may become saturated when the ice melts, giving rise to thaw settlement and loss of bearing capacity. Repeated cycles of freezing and thawing change the structure of the soil, again reducing its bearing capacity. Rigid concrete pavements are more able to resist frost action than flexible bituminous pavements.

Geohazards obviously have an adverse influence on roads. Movement of sand in arid areas can bury obstacles in its path such as routeways. Such moving sand necessitates continuous and often costly maintenance activities. In addition, the high rates of evaporation in hot arid areas may lead to ground heave due to the precipitation of minerals within the capillary fringe of the soil. In the absence of downward leaching, surface deposits become contaminated with precipitated salts, particularly sulphates and chlorides. Landslides on either natural or man-made slopes adversely affect roadways (Al-Homoud and Tubeileh, 1997). Maerz et al. (2005) discussed rockfall hazard rating systems in relation to the protection of highways in Missouri. Slope stabilization measures have been dealt with earlier. Not only can flooding

Figure 9.23

Erosion and removal of part of the approach road to the Mvoti Bridge, Natal, South Africa.

disrupt road traffic, but it can cause the destruction of roads (Bell, 1994a; Fig. 9.23). Earthquake damage to routeways can cause disruption to urban centres that rely on these routeways (Fig. 9.24). Damage to a particular zone of a routeway can affect an area extending beyond the zone. Geological conditions, especially soil properties, potential relative ground displacement and potential horizontal and vertical strain distribution therefore must be taken into account when designing routeways in seismically active regions. Notable ground movements can result from mining subsidence, the type of movements and the time of their occurrence being influenced by the method of mining used. It probably will be necessary to fill mined voids beneath roads with bulk grout. Faults and dykes in mining areas can concentrate the effects of mining subsidence, giving rise to surface cracking or the development of steps, which can lead to severe surface disruption of highways (Stacey and Bell, 1999). Such movement entails local resurfacing of highways. Natural voids and cavities in rock masses also can represent potential subsidence problems in routeway construction (Fig. 9.25). The collapse of a sinkhole beneath a road can be responsible for disastrous consequences, for example, Boyer (1997) referred to vehicles falling into newly opened sinkholes in Maryland and the death or serious injury of the occupants. Once sinkholes have been located, they are filled with bulk grout (Petersen et al., 2003). Geogrids are now used in road construction in areas where subsidence, due either to natural causes or mining, could pose a future threat.

Figure 9.24

Luanhe Bridge on the Beijing–Yuguan Highway, collapse of the deck and piers due to the Tangshan earthquake, 1976.

Figure 9.25

Collapse of a road over a sinkhole, Centurian, South Africa.

In the case of cavity collapse, they hopefully prevent the road falling into the cavity before repairs are carried out (Cooper and Saunders, 2002).

Soil Stabilization and Road Construction

The objectives of mixing additives with soil are to improve volume stability, strength and stress–strain properties, permeability and durability. In clay soils, swelling and shrinkage can be reduced. Good mixing of stabilizers with soil is the most important factor affecting the quality of results. Cement and lime are the two most commonly used additives.

The principal use of soil–cement is as a base material underlying the pavement (Bell, 1995). One of the reasons soil–cement is used as a base is to prevent pumping of fine-grained subgrade soils into the pavement above. The thickness of the soil–cement base depends on subgrade strength, pavement design, traffic and loading conditions and thickness of the wearing surface. Frequently, however, soil-cement bases are around 150–200 mm in thickness. The principal use of the addition of lime to soil is for subgrade and sub-base stabilization, and as a construction expedient on wet sites where lime is used to dry out the soil. Lime stabilization of expansive clay soils, prior to construction, can minimize the amount of shrinkage and swelling they undergo (Bell, 1996a). However, significant SO_4 content (i.e. in excess of 5000 mg kg^{-1}) in clay soils can mean that it reacts with CaO to form ettringite $[Ca_6Al_2(OH)_{12}(SO_4)_3 \cdot 27H_2O]$ or thaumasite $[CaSiO_3 \cdot CaCO_3 \cdot CaSO_4 \cdot 14 \cdot 5H_2O]$ with resultant expansion and heave, as happened during the construction of a motorway in Oxfordshire, England. The main sources of sulphate likely to cause heave in lime-stabilized clay soils beneath roads are gypsum and pyrite. Nevertheless, Wild et al. (1999) claimed that such swelling in lime-stabilized soils can be suppressed by the use of ground-granulated blast-furnace slag, GGBS. They recommended that 60–80% of the lime should be replaced by GGBS in order to minimize or eliminate sulphate expansion, and that compaction should be wet of optimum. Similarly, Kumar and Sharma (2004) indicated that the addition of fly ash to lime or cement would, on stabilization, reduce swelling and improve the engineering characteristics of expansive soils. As far as cement and lime stabilization for roadways is concerned, stabilization is brought about by the addition of between 3 and 6% of cement or lime (by dry weight of soil). Subgrade stabilization involves stabilizing the soil in place or stabilizing borrow materials that are used for sub-bases. After the soil, which is to be stabilized, has been brought to grade, the roadway should be scarified to full depth and width and then partly pulverized. A rooter, grader, scarifier and/or disc-harrow are used for initial scarification, followed by a rotary mixer for pulverization. After mixing in the cement or lime and any additional water needed to reach the optimum moisture content, compaction and grading to the final level is carried out (Fig. 9.26). Finally, the processed layer is covered with a water-proof membrane, commonly bitumen emulsion, to prevent drying out and to ensure hydration.

Figure 9.26

Lime stabilization for an access road at Heathrow airport, London, England.

The properties developed by compacted cement or lime-stabilized soils are governed by the amount of cement or lime added on the one hand and compaction on the other. With increasing cement or lime content, the strength and bearing capacity increase, as does the durability to wet–dry cycles. The permeability generally decreases but tends to increase in clayey soils.

The Use of Geotextiles in Road Construction

The improvement in the performance of a pavement attributable to the inclusion of geotextiles comes mainly from their separation and reinforcing functions. This can be assessed in terms of either an improved system performance (e.g. reduction in deformation or increase in traffic passes before failure) or reduced aggregate thickness requirements (where reductions of the order 25–50% are feasible for low-strength subgrade conditions with suitable geotextiles).

The most frequent role of geotextiles in road construction is as a separator between the sub-base and subgrade. This prevents the subgrade material from intruding into the sub-base due to repeated traffic loading and so increases the bearing capacity of the system. The savings in sub-base materials, which would otherwise be lost due to mixing with the subgrade, can sometimes cover the cost of the geotextile. The range of gradings or materials that can be used as sub-bases with geotextiles normally is greater than when they are not used. Nevertheless, the sub-base materials preferably should be angular, compactable and sufficiently well graded to provide a good riding surface.

If a geotextile is to increase the bearing capacity of a subsoil or pavement significantly, then large deformations of the soil-geotextile system generally must be accepted, as a geotextile has no bending stiffness, is relatively extensible, usually is laid horizontally and is restrained from extending laterally. Thus, considerable vertical movement is required to provide the necessary stretching to induce the tension that affords vertical load-carrying capacity to the geotextile. Therefore, geotextiles are likely to be of most use when included within low-density sands and very soft clays. Although large deformations may be acceptable for access and haul roads, they are not acceptable for most permanent pavements. In this case, the geotextile at the sub-base, subgrade interface should not be subjected to mechanical stress or abrasion. When geotextile is used in temporary or permanent road construction, it helps redistribute the load above any local soft spots that occur in a subgrade. In other words, the geotextile deforms locally and progressively redistributes load to the surrounding areas, thereby limiting local deflections. As a result, the extent of local pavement failure and differential settlement is reduced.

The use of geogrid reinforcement in road construction helps restrain lateral expansive movements at the base of the aggregate. This gives rise to improved load redistribution in the sub-base that, in turn, means a reduced pressure on the subgrade. In addition, the cyclic strains in the pavement are reduced. The stiff load bearing platform created by the interlocking of granular fill with geogrids is utilized effectively in the construction of roads over weak soil. Reduction of 30–50% in the required aggregate thickness may be achieved. Geogrids can be used within a granular capping layer when constructing roads over variable sub-grades. They also have been used to construct access roads across peat, the geogrid enabling the roads to be “floated” over the surface.

In arid regions, impermeable geomembranes can be used as capillary breaks to stop the upward movement of salts where they would destroy the road surface. Geomembranes also can be used to prevent the formation of ice lenses in permafrost and other frost-prone regions. The geomembrane must be located below the frost line and above the water table.

Where there is a likelihood of uplift pressure disturbing a road constructed below the piezometric level, it is important to install a horizontal drainage blanket. This intercepts the rising water and conveys it laterally to drains at the side of the road. A geocomposite can be used for effective horizontal drainage. Problems can arise when sub-bases are sensitive to moisture changes, that is, they swell, shrink or degrade. In such instances, it is best to envelop the sub-base in a geomembrane; or excavate, replace and compact the upper layers of the sub-base in an envelope of impermeable geomembrane.

Embankments

The engineering properties of soils used for embankments, such as their shear strength and compressibility, are influenced by the amount of compaction they have undergone. Accordingly, the desired amount of compaction is established in relation to the engineering properties required for the embankment to perform its design function. A specification for compaction needs to indicate the type of compaction equipment to be used, its mass, speed and travel, and any other factors influencing performance such as frequency of vibration, thickness of layers to be compacted and number of passes of the compactor (Table 9.8).

Embankments are mechanically compacted by laying and rolling soil in thin layers. The soil particles are packed more closely due to a reduction in the volume of the void space, resulting from the momentary application of loads such as rolling, tamping or vibration. Compaction involves the expulsion of air from the voids without the moisture content being changed significantly. Hence, the degree of saturation is increased. However, all the air cannot be expelled from the soil by compaction so that complete saturation is not achievable. Nevertheless, compaction does lead to a reduced tendency for changes in the moisture content of the soil to occur. The method of compaction used depends on the soil type, including its grading and moisture content at the time of compaction; the total quantity of material, layer thickness and rate at which it is to be compacted; and the geometry of the proposed earthworks.

A clayey soil is stiff and therefore more difficult to compact when the moisture content is low. As the moisture content increases, it enhances the interparticle repulsive forces, thus separating the particles causing the soil to soften and become more workable. This gives rise to higher dry densities and lower air contents. As saturation is approached, however, pore water pressure effects counteract the effectiveness of the compactive effort. Each soil therefore has an optimum moisture content at which the soil has a maximum dry density. The compaction characteristics of clay are governed largely by its moisture content. For instance, a greater compactive effort is necessary as the moisture content is lowered. It may be necessary to use thinner layers and more passes by a heavier compaction plant than required for granular materials. The properties of cohesive fills also depend to a much greater extent on the placement conditions than do those of a coarse-grained fills (Charles and Skinner, 2001). In addition, the shear strength and compressibility of compacted clayey soil depend on its density and moisture content, and are influenced by the pore water pressure. Compaction of cohesive soils should be carried out when the moisture content of the soil is

Table 9.8. Typical compaction characteristics for soils used in earthwork construction (after Anon, 1981b). With kind permission of the British Standards Institutions

Soil	Major group	Subgroup	Suitable type of compaction plant	Minimum number of passes for satisfactory compaction	Maximum thickness of compacted layer (mm)	Remarks
Coarse soils	Gravels and gravelly soils	Well-graded gravel and gravel/sand mixtures; little or no fines	Grid roller over 540 kg per 100 mm of roll Pneumatic-tyred roller over 2000 kg per wheel Vibratory plate compactor over 1100 kgm^{-2} of baseplate Smooth-wheeled roller Vibro-rammer Self-propelled tamping roller	3–12, depending on type of plant	75–275, depending on type of plant	
		Well-graded gravel/sand mixtures with excellent clay binder				
		Uniform gravel; little or no fines				
		Poorly graded gravel and gravel/sand mixtures; little or no fines				
		Gravel with excess fines, silty gravel, clayey gravel, poorly graded gravel/sand/clay mixtures				
	Sands and sandy soils	Well-graded sands and gravelly sands; little or no fines				
		Well-graded sands with excellent clay binder				
	Uniform sands and gravels	Uniform gravels; little or no fines	Smooth-wheeled roller below 500 kg per 100 mm of roll Grid roller below 540 kg per 100 mm	3–16, depending on type of plant	75–300, depending on type of plant	
		Uniform sands; little or no fines				

		Poorly graded sands; little or no fines Sands with fines; silty sands, clayey sands, poorly graded sand/clay mixtures	Pneumatic-tyred roller below 1500 kg per wheel Vibrating roller Vibrating plate compactor Vibro-tamper			
Fine soils	Soils having low plasticity	Silts (inorganic) and very fine sands, silty or clayey fine sands with slight plasticity Clayey silts (inorganic) Organic silts of low plasticity	Sheepsfoot roller Smooth-wheeled roller Pneumatic-tyred roller Vibratory roller over 70 kg per 100 mm roll Vibratory plate compactor over 1400 kg m^{-2} of baseplate Vibro-tamper Power rammer	4–8, depending on type of plant	100–450, depending on type of plant	If moisture content is low, it may be preferable to use a vibratory roller Sheepsfoot rollers are best suited to soils at a moisture content below their plastic limit
	Soils having medium plasticity	Silty and sandy clays (inorganic) of medium plasticity Clays (inorganic) of medium plasticity				
	Soils having high plasticity	Fine sandy and silty soils, plastic silts Clays (inorganic) of high plasticity, fat clays				Should only be used when circumstances are favourable

Note: Organic clays generally are unsuitable for earth works and those of high plasticity should not be used.

not more than 2% above the plastic limit. If it exceeds this figure, the soil should be allowed to dry. Over-compaction of soil on site, that is, compacting the soil beyond the optimum moisture content, means that the soil becomes softer. As far as granular material is concerned, it can be compacted at its natural moisture content, and normally it is easier to compact. When compacted, granular soils have a high load-bearing capacity and are not compressible. Usually, they are not susceptible to frost action unless they contain a high proportion of fines. Unfortunately, if granular material contains a significant amount of fines, then high pore water pressures can develop during compaction if the moisture content of the soil is high. Some soils such as highly plastic and organic clays undergo large volume changes and cannot be stabilized effectively by compaction.

The most critical period during the construction of an embankment is just before it is brought to grade or shortly thereafter. At this time, pore water pressure, due to consolidation in the embankment and foundation, is at a maximum. The magnitude and distribution of pore water pressure developed during construction depend primarily on the construction water content, the properties of the soil, the height of the embankment and the rate at which dissipation by drainage can occur. Water contents above optimum can cause high construction pore water pressure, which increase the danger of rotational slips in embankments constructed of clayey soil (Aubeny and Lytton, 2004). Well graded clayey sands and sand–gravel–clay mixtures develop the highest construction pore water pressures, whereas uniform silts and fine silty sands are the least susceptible.

Geogrids or geomats can be used in the construction of embankments over poor ground without the need excavate the ground and substitute granular fill. They can allow acceleration of fill placement often in conjunction with vertical band drains. Layers of geogrids or geowebs can be used at the base of an embankment to intersect potential deep failure surfaces or to help construct an embankment over peat deposits. Geogrids also can be used to encapsulate a drainage layer of granular material at the base of an embankment. The use of band drains or a drainage layer helps reduce and regulate differential settlement. A geocell mattress can be constructed at the base of an embankment that is to be constructed over soft soil. The cells are generally about 1 m high and are filled with granular material. This also acts as a drainage layer. The mattress intersects potential failure planes, and its rigidity forces them deeper into firmer soil. The rough interface at the base of the mattress ensures mobilization of the maximum shear capacity of the foundation soil and significantly increases stability. Differential settlement and lateral spread are minimized.

If the soil beneath a proposed embankment, which is to carry a road, is likely to undergo appreciable settlement, then the soil can be treated. One of the commonest forms of treatment is precompression. Those soils that are best suited to improvement by precompression include compressible silts, saturated soft clays, organic clays and peats. The presence of thin layers of sand or silt in compressible material may mean that rapid consolidation takes place. Unfortunately, this may be accompanied by the development of abnormally high pore water pressures in those layers beyond the edge of the precompression load. This ultimately lowers their shearing resistance to less than that of the surrounding weak soil. Such excess pore water pressures may have to be relieved by vertical drains.

Precompression normally is brought about by preloading, which involves the placement and removal of a dead load. This compresses the foundation soils, thereby inducing settlement prior to construction. If the load intensity from the dead weight exceeds the pressure imposed by the final load of the embankment, this is referred to as surcharging. In other words, the surcharge is the excess load additional to the final load and is used to accelerate the compression process. The surcharge load is removed after a certain amount of settlement has taken place. Soil undergoes considerably more compression during the first phase of loading than during any subsequent reloading. Moreover, the amount of expansion following unloading is not significant (Alonso et al., 2000). The installation of vertical drains (e.g. sandwicks or band drains) beneath the precompression load helps shorten the time required to bring about primary consolidation. The water from the drains flows into a drainage blanket placed at the surface or to highly permeable layers deeper in the soil. Another method of bringing about precompression is by vacuum preloading by pumping from beneath an impervious membrane placed over the ground surface (Tang and Shang, 2000).

Reinforced Earth

Reinforced earth is a composite material consisting of soil that is reinforced with metallic or geogrid strips. The effectiveness of the reinforcement is governed by its tensile strength and the bond it develops with the surrounding soil. It also is necessary to provide some form of barrier to contain the soil at the edge of a reinforced earth structure (Fig. 9.8). This facing can be either flexible or stiff, but it must be strong enough to retain the soil and to allow the reinforcement to be fixed to it. As reinforced earth is flexible and the structural components are built at the same time as backfill is placed, it is particularly suited for use over compressible foundations where differential settlements may occur during or soon after construction. In addition, as a reinforced earth wall uses small prefabricated components and requires no formwork, it can be adapted to the required variations in height or shape. Granular fill is the most suitable in that it is free draining and non-frost susceptible, as well as being virtually non-corrosive as far as the reinforcing elements are concerned. It also is relatively stable, eliminating post-construction movements. Nonetheless, fine-grained materials can be used as fill but a slower construction schedule is necessary.

Railroads

Railroads have played and continue to play an important role in national transportation systems, although the construction of new railroads on a large scale is something that belongs to the past. Nonetheless, railroads continue to be built such as those associated with high-speed networks. A vital part of a high-speed railroad, with trains travelling at speeds of up to 300 km h^{-1}, is the trackbed support. In other words, the dynamic behaviour of foundations and earthworks involves a detailed understanding of the soil–structure interaction. This distinguishes a modern high-speed railway from other railways or highways. Obviously, the grades and curvature of railroads impose stricter limits than do those associated with highways. Furthermore, underground systems are being and will continue to be constructed beneath many large cities in order to convey large numbers of people from one place to another quickly and efficiently.

Topography and geology are as important in railroad construction as in highway construction (O'Riordan, 2003). A good illustration of this has been provided by Baynes et al. (2005), who outlined an engineering geological and geomorphological approach to the construction of railways in the Pilbara, Western Australia. As noted, a very stable trackbed and earthworks are necessary for a high-speed railroad. Accordingly, drainage is an important aspect of trackbed design, so that surface water is removed efficiently and groundwater level is maintained below the subgrade. Another important factor is trackbed stiffness. Consequently, where sudden changes of stiffness are likely to occur, notably between embankments and bridges, this can involve the use of layers of cement-stabilized and well-graded granular material to provide a gradual transition in stiffness. Deep dry soil mixing can be used where embankments are constructed over soft clays and peats.

Railroads obviously can be affected by geohazards. In rugged terrain, in particular, trains may be interrupted for a time by rock falls, landslides or mudflows (Fig. 9.27). Areas prone to such hazards along a railroad need to be identified and, where possible, stabilization or protective measures carried out (see Chapter 3). In some areas, because of the nature of the terrain, it may be impossible to stabilize entire slopes. In such cases, warning devices can be installed that are triggered by such movements and cause trains to halt. Be that as it may, lengths of track that are subjected to mass movements should be inspected regularly. Remedial works in landslipped areas can include some combination of subsurface drainage, redesign and/or reinforcement of slopes. Railway services also may be interrupted seriously by flooding, earthquakes or the development of sinkholes (Erwin and Brown, 1988).

Railway track formations normally consist of a layer of coarse aggregate, the ballast, in which the sleepers are embedded. The ballast may rest directly on the subgrade or, depending on the bearing capacity, on a layer of blanketing sand. The function of the ballast is to provide a free-draining base that is stable enough to maintain the track alignment with the minimum of maintenance. The blanketing sand provides a filter that prevents the migration of fines from

Figure 9.27

A mudflow partially engulfing a train on the Chengdu–Kunming railway, China.

the subgrade into the ballast due to pumping. The ballast must be thick enough to retain the track in position and to prevent intermittent loading from deforming the subgrade, and the aggregate beneath the sleepers must be able to resist abrasion and attrition. Hence, strong, good–wearing angular aggregates are required, such as provided by many dense siliceous rocks. The thickness of the ballast can vary from as low as 150 mm for lightly trafficked railroads up to 500 mm on railroads that carry high-speed trains or heavy traffic. The blanketing layer of sand normally has a minimum thickness of 150 mm.

Under repeated loading, differential permanent strains develop in the ballast of a rail track that bring about a change in the rail line and level. If the voids in the ballast are allowed to fill with fine-grained material, then a failure condition can develop. The fines may be derived by pumping from the sub-ballast or subgrade if these become saturated. Accordingly, railway track ballast requires regular attention to maintain line and level.

Bridges

As with tunnels, the location of bridges may be predetermined by the location of the routeways of which they form part. Consequently, this means that the ground conditions beneath bridge locations must be adequately investigated. This is especially the case when a bridge has to cross a river (Nichol and Wilson, 2002). The geology beneath a river should be correlated with the geology on both banks, and drilling beneath the river should go deep enough to determine the solid rock in place. The geological conditions may be complicated by the presence of a buried channel beneath a river. Buried channels generally originated during the Pleistocene epoch when valleys were deepened by glacial action, and sea levels were at lower positions. Subsequently, these channels were occupied by various types of sediments, which may include peat. The data obtained from a site investigation should enable the bridge, piers and abutments to be designed satisfactorily.

The ground beneath bridge piers has to support not only the dead load of the bridge but also the live load of the traffic that the bridge will carry, in addition to accommodating the horizontal thrust of the river water when bridges cross rivers. The choice of foundations usually is influenced by a number of factors. For example, the existence of sound rock near the surface allows spread foundations to be used without the need for widespread piling, whereas piled foundations are adopted for flood plains and rivers where alluvial deposits overlie bedrock (Kitchener and Ellison, 1997). Because of the strong currents and the high tidal range in an estuary, precast concrete open-bottom caissons, which are floated out and put in place by specially adapted barges, may be used for piers. Such caissons provide permanent formwork shells for the concrete infill. Alternatively, piers can be designed as cellular structures supported by cylindrical caissons.

The anchorages for suspension bridges have to resist very high pull-out loads. For example, the Hessle anchorage, on the north side of the river, for the Humber Bridge, England, has to resist a horizontal pull of 38,000 tonnes. Resistance is derived from friction at the soil or rock/concrete interface at the base of the anchorage, from the passive resistance at the front and from wedge action at the sides.

When a bridge is constructed across a river, the effective cross-sectional area of the river is reduced by the piers, which leads to an increase in its velocity of flow. This and the occurrence of eddies around the piers enhances scouring action. Less scouring generally takes place where a river bed is formed of cobbles and gravels than where it is formed of sand or finer-grained material. Scouring of river bed materials around bridge piers has caused some bridges to fail. During floods, damage is caused by very high peak flows, by build-up of debris at the bridge and by excessive scour around supporting caissons/piers (Fig. 8.9). The problem of scouring is accentuated in estuaries, especially where the flow patterns of the ebb and flood tides are different.

Bridges obviously are affected by ground movements such as subsidence. Subsidence movements can cause relative displacements in all directions and therefore subject a bridge to tensile and compressive stresses. Although a bridge can have a rigid design to resist such ground movements, it usually is more economic to articulate it, thereby reducing the effects of subsidence.In the case of multi-span bridges, the piers should be hinged at the top and bottom to allow for tilting or change in length. Jacking sockets can be used to maintain the level of the deck. As far as shallow, abandoned room and pillar workings are concerned, it usually is necessary to fill voids beneath a bridge with grout.

Seismic forces in earthquake-prone regions can cause damage to bridges and must therefore be considered in bridge design (Fig. 9.24). Most seismic damage to low bridges has been caused by failures of substructures resulting from large ground deformation or liquefaction (Kubo and Katayama, 1978). Indeed, it appears that the worst damage is sustained by bridges located on soft ground, especially that capable of liquefaction. Failure or subsidence of backfill in a bridge approach, leading to an abrupt change in profile, can prevent traffic from using the approach even if the bridge is undamaged. Such failure frequently exerts large enough forces on abutments to cause damage to substructures. On the other hand, seismic damage to superstructures due purely to the effects of vibrations is rare. Nonetheless, as a result of substructure failure, damage can occur within bearing supports and hinges, which combined with excessive movement of substructures, can bring about the collapse of a superstructure. By contrast, the effects of vibrations can be responsible for catastrophic failures of high bridges that possess relatively little overall stiffness. Arch-type bridges are the strongest, whereas simple or cantilever beam type bridges are the most vulnerable to seismic effects. Furthermore, the greater the height of substructures and the greater the number of spans, the more likely is a bridge to collapse.

Foundations for Buildings

Types of Foundation Structure

The design of foundations embodies three essential operations, namely, calculating the loads to be transmitted by the foundation structure to the soils or rocks supporting it, determining the engineering performance of these soils and rocks, and then designing a suitable foundation structure.

Footings distribute the load to the ground over an area sufficient to suit the pressures to the properties of the soil or rock. Their size therefore is governed by the strength of the foundation materials. If the footing supports a single column, it is known as a spread or pad footing, whereas a footing, beneath a wall is referred to as a strip or continuous footing.

The amount and rate of settlement of a footing due to a given load per unit area of its base is a function of the dimensions of the base, and of the compressibility and permeability of the foundation materials located between the base and a depth that is at least one and a half times the width of the base. If footings are to be constructed on cohesive soil, it is necessary to determine whether or not the soil is likely to swell or shrink according to any seasonal variations. Fortunately, significant variations below a depth of about 2 m are rather rare.

Footings usually provide the most economical type of foundation structure, but the allowable bearing capacity must be available to provide an adequate factor of safety against shear failure in the soil and to ensure that settlements are not excessive. Settlement for any given pressure increases with the width of footing in almost direct proportion on clays and to a lesser degree on sands.

A raft permits the construction of a satisfactory foundation in materials whose strength is too low for the use of footings. The chief function of a raft is to spread the building load over as great an area of ground as possible and thus reduce the bearing pressure to a minimum. In addition, a raft provides a degree of rigidity that reduces differential movements in the superstructure. The settlement of a raft foundation does not depend on the weight of the building that is supported by the raft. It depends on the difference between this weight and the weight of the soil that is removed prior to the construction of the raft, provided the heave produced by the excavation is inconsequential. A raft can be built at a sufficient depth so that the weight of soil removed equals the weight of the building. Hence, such rafts are sometimes called floating foundations. The success of this type of foundation structure in overcoming difficult soil conditions has led to the use of deep raft and rigid frame basements for a number of high buildings on clay.

When the soil immediately beneath a proposed structure is too weak or too compressible to provide adequate support, the loads can be transferred to more suitable material at greater depth by means of piles. Such bearing piles must be capable of sustaining the load with an adequate factor of safety, without allowing settlement detrimental to the structure to occur. Although these piles derive their carrying capacity from end bearing at their bases, the friction along their sides also contributes towards this. Indeed, friction is likely to be the predominant factor for piles in clays and silts, whereas end bearing provides the carrying capacity for piles terminating in gravel or rock.

Piles may be divided into three main types according to the effects of their installation, namely, displacement piles, small-displacement piles and non-displacement piles. Displacement piles are installed by driving and so their volume has to be accommodated below ground by vertical and lateral displacements of soil that may give rise to heave or compaction, which may have detrimental effects on any neighbouring structures. Driving also may cause piles that are already installed to lift. Driving piles into clay may affect its consistency. In other words, the penetration of the pile, combined with the vibrations set up by the falling hammer, destroy the structure of the clay and initiate a new process of consolidation that drags the piles in a downward direction, indeed they may settle on account of their contact with the remoulded mass of clay even if they are not loaded. Sensitive clays are affected in this way, whereas insensitive clays are not. Small displacement piles include some piles that may be used in soft alluvial ground of considerable depth. They also may be used to withstand uplift forces. They are not suitable in stiff clays or gravels. Non-displacement piles are formed by boring and the hole may be lined with casing that is or is not left in place. When working near existing structures that are founded on loose sands or silts, particularly if these are saturated, it is essential to avoid the use of methods that cause dangerous vibrations that may give rise to a quick condition.

For practical purposes, the ultimate bearing capacity may be taken as that load which causes the head of the pile to settle 10% of the pile diameter. The ratio between the settlement of a pile foundation and that of a single pile acted upon by the design load can have almost any value. This is due to the fact that the settlement of an individual pile depends only on the nature of the soil in direct contact with the pile, whereas the settlement of a pile foundation also depends on the number of piles and on the compressibility of the soil located between the level of the tips of the piles and the surface of the bedrock.

Bearing Capacity

Foundation design is concerned primarily with ensuring that movements of a foundation are kept within limits that can be tolerated by the proposed structure without adversely affecting its functional requirements. Hence, the design of a foundation structure requires an understanding of the local geological and groundwater conditions and, more particularly, an appreciation of the various types of ground movement that can occur.

In order to avoid shear failure or substantial shear deformation of the ground, the foundation pressures used in design should have an adequate factor of safety when compared with the ultimate bearing capacity of the foundation. The ultimate bearing capacity is the value of the loading intensity that causes the ground to fail in shear. If this is to be avoided, then a factor of safety must be applied to the ultimate bearing capacity, the value obtained being the maximum safe bearing capacity. But even this value may still mean that there is a risk of excessive or differential settlement. Thus, the allowable bearing capacity is the value that is used in design, also taking into account all possibilities of ground movement, and so its value is normally less than that of the safe bearing capacity. The value of ultimate bearing capacity depends on the type of foundation structure as well as on the soil properties. For example, the dimensions, shape and depth at which a footing is placed all influence the bearing capacity. More specifically, the width of a foundation is important in sands;

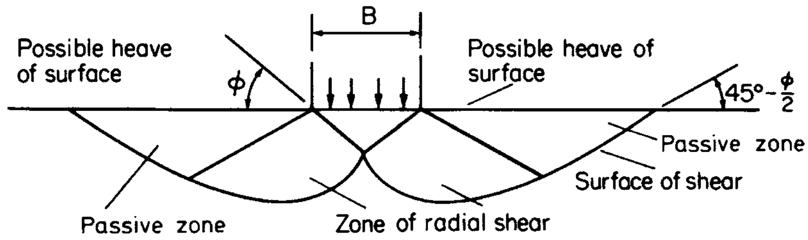


Figure 9.28

Foundation failure.

the greater the width, the larger the bearing capacity, whereas it is of little effect in saturated clays. With uniform soil conditions, the ultimate bearing capacity increases with depth of installation of the foundation structure. This increase is associated with the confining effects of the soil, the decreased overburden pressure at foundation level and with the shear forces that can be mobilized between the sides of the foundation structure and the ground.

There are usually three stages in the development of a foundation failure. Firstly, the soil beneath the foundation is forced downwards in a wedge-shaped zone (Fig. 9.28). Consequently, the soil beneath the wedge is forced downwards and outwards, elastic bulging and distortion taking place within the soil mass. Secondly, the soil around the foundation perimeter pulls away from the foundation, and the shear forces propagate outward from the apex of the wedge. This is the zone of radial shear in which plastic failure by shear occurs. Thirdly, if the soil is very compressible or can endure large strains without plastic flow, then the failure is confined to fan-shaped zones of local shear. The foundation displaces downwards with little load increase. On the other hand, if the soil is more rigid, the shear zone propagates outward until a continuous surface of failure extends to ground surface and the surface heaves.

The weight of the material in the passive zone resists the lifting force and provides the reaction through the other two zones that counteract the downward motion of the foundation structure (Fig. 9.28). Hence, the bearing capacity is a function of the resistance to the uplift of the passive zone. This, in turn, varies with the size of the zone (which is a function of the internal angle of friction), with the unit weight of the soil and with the sliding resistance along the lower surface of the zone (which is a function of the cohesion, internal angle of friction and unit weight of the soil). A surcharge placed on the passive zone or an increase in the depth of the foundation therefore increases the bearing capacity.

The stress distribution due to a structure declines rapidly with depth within the soil. It should be determined in order to calculate the bearing capacity and settlement at given depths.

The pressure acting between the bottom of a foundation structure and the soil is the contact pressure. The assumption that a uniformly loaded foundation structure transmits the load uniformly so that the ground is uniformly stressed is by no means valid. For example, the intensity of the stresses at the edges of a rigid foundation structure on hard clay is theoretically infinite. In fact, the clay yields slightly and so reduces the stress at the edges. As the load is increased, more and more local yielding of the ground material takes place until, when the loading is close to that which would cause failure, the distribution is probably very nearly uniform. Therefore, at working loads, a uniformly loaded foundation structure on clay imposes a widely varying contact pressure. On the other hand, a rigid footing on the surface of dry sand imposes a parabolic distribution of pressure. Since there is no cohesion in such material, no stress can develop at the edges of a footing. If the footing is below the surface of the sand, then the pressure at the edge is no longer zero but increases with depth. The pressure distribution tends therefore to become more nearly uniform as the depth increases. If a footing is perfectly flexible, then it will distribute a uniform load over any type of foundation material.

The ultimate bearing capacity of foundations on coarse soils depends on the width and depth of placement of the foundation structure as well as the angle of shearing resistance. The position of the water table in relation to the foundation structure has an important influence on the ultimate bearing capacity. High groundwater levels lower the effective stresses in the ground, so that the ultimate bearing capacity is reduced by anything up to 50%. Generally speaking, gravels and dense sands afford good foundations. It is possible to estimate the bearing capacity of such soils from plate load tests or penetration tests. However, loosely packed sands are likely to undergo settlement when loaded.

The ultimate bearing capacity of foundations on clay soils depends on the shear strength of the soil and the shape and depth at which the foundation structure is placed. The shear strength of clay is, in turn, influenced by its consistency. Although there is a small decrease in the moisture content of clay beneath a foundation structure, which gives rise to a small increase in soil strength, this is of no importance as far as estimation of the factor of safety against shear is concerned. Saturated clays in relation to applied stress behave as cohesive materials provided that no change of moisture content occurs. Thus, when a load is applied to saturated clay, it produces excess pore water pressures that are not dissipated quickly. In other words, the angle of shearing resistance is equal to zero. The assumption that $\phi = 0$ forms the basis of all normal calculations of the ultimate bearing capacity in clays (Skempton, 1951). The strength then may be taken as the undrained shear strength or one half the unconfined compressive strength. To the extent that consolidation does occur, the results of analyses based on the premise that $\phi = 0$ are on the safe side. Only in special cases, with prolonged loading periods or with very silty clays, is the assumption sufficiently far from the truth to justify a more elaborate analysis.

For all types of foundation structures on clays, the factors of safety must be adequate against bearing capacity failure. Generally speaking, experience has indicated that it is desirable to use a factor of safety of 3; yet, although this means that complete failure almost invariably is ruled out, settlement may still be excessive. It therefore is necessary to give consideration to the settlement problem if bearing capacity is to be viewed correctly. More particularly, it is important to make a reliable estimate of the amount of differential settlement that may be experienced by a structure. If the estimated differential settlement is excessive, it may be necessary to change the layout or type of foundation structure.

If a rock mass contains few defects, the allowable contact pressure at the surface may be taken conservatively as the unconfined compressive strength of the intact rock. Most rock masses, however, are affected by joints or weathering that may significantly alter their strength and engineering behaviour. The great variation in the physical properties of weathered rock and the non-uniformity of the extent of weathering, even at a single site, permit few generalizations concerning the design and construction of foundation structures. The depth to bedrock and the degree of weathering must be determined. If the weathered residuum plays the major role in the regolith, rock fragments being of minor consequence, then the design of rafts or footings should be according to the matrix material. Piles can provide support at depth.

Settlement

The average values of settlement beneath a structure, together with the individual settlements experienced by its various parts, influence the degree to which the structure serves its purpose. The damage attributable to settlement can range from complete failure of the structure to slight disfigurement (Fig. 9.29).

If coarse soils are densely packed, then they are almost incompressible. For example, recorded settlements for footings on coarse soils often are of the order of 25 mm or less and rarely exceed 50 mm. In fact, the commonly accepted basis of design is that the total settlement of a footing should be restricted to about 25 mm, as by so doing the differential settlement between adjacent footings is confined within limits that can be tolerated by a structure. Loosely packed sand located above the water table undergoes some settlement but is otherwise stable. Greater settlement is likely to be experienced where foundation level is below the water table. Additional settlement may occur if the water table fluctuates or the ground is subjected to vibrations. Settlement commonly is relatively rapid, but there can be a significant time lag when stresses are large enough to produce appreciable grain fracturing. Nonetheless, settlement in sands and gravels frequently is substantially complete by the end of the construction period.

Figure 9.29

Settlement of a building on clay, Goteborg, Sweden. Note the window frames are twisted somewhat and the lower ones boarded up; the shearing in the brickwork, especially beneath the second floor windows; and the slope on the down-comer.

Settlement can present a problem in clayey soils, so that the amount that is likely to take place when they are loaded needs to be determined. Settlement invariably continues after the construction period, often for several years. Immediate or elastic settlement is that which occurs under constant-volume (undrained) conditions when clay deforms to accommodate the imposed shear stresses. Primary consolidation in clay takes place due to the void space being gradually reduced as the pore water and/or air are expelled therefrom on loading. The rate at which this occurs depends on the rate at which the excess pore water pressure, induced by a structural load, is dissipated, thereby allowing the structure to be supported entirely by the soil skeleton. Consequently, the permeability of the clay is all important. After sufficient time has elapsed, excess pore water pressures approach zero, but a deposit of clay may continue to decrease in volume. This is referred to as secondary consolidation and involves compression of the soil fabric.

Settlement is rarely a limiting condition in foundations on most fresh rocks. Consequently, it does not entail special study except in the case of special structures where settlements must be small. The problem then generally resolves itself into one of reducing the unit–bearing load by widening the base of a structure or using spread footings. In some cases, appreciable differential settlements are provided for by designing articulated structures capable of taking differential movements of individual sections without damaging the structure. Severe settlements, however, may take place in low grade compaction shale.

Generally, uniform settlements can be tolerated without much difficulty, but large settlements are inconvenient and may cause serious disturbance to services, even where there is no evident damage to the structure. However, differential settlement is of greater significance than maximum settlement since the former is likely to distort or even shear a structure. Buildings that suffer large maximum settlement also are likely to experience large differential settlement. Therefore, both should be avoided.

Burland and Wroth (1975) accepted a safe limit for angular distortion (difference in settlement between two points) of 1:500 as satisfactory for framed buildings, but stated that it was unsatisfactory for buildings with load-bearing walls. Damage in the latter has occurred with very much smaller angular distortions. The rate at which settlement occurs also influences the amount of damage suffered.

For most buildings, it is the relative deflections that occur after completion that cause damage. Therefore, the ratio between the immediate and total settlement is important. In overconsolidated clays, this averages about 0.6, whereas it usually is less than 0.2 for normally consolidated clay. This low value coupled with larger total settlement makes the problems of design for normally consolidated clays much more demanding than for overconsolidated clays.

Settlements may be reduced by the correct design of the foundation structure. This may include larger or deeper foundations. Also, settlements can be reduced if the site is preloaded or surcharged prior to construction or if the soil is subjected to dynamic compaction or vibrocompaction. It is advantageous if the maximum settlement of large structures is reached earlier than later. The installation of sandwicks or band drains, which provide shorter drainage paths for the escape of water to strata of higher permeability, is one means by which this can be achieved. Sandwicks and band drains may effect up to 80% of the total settlement in cohesive soils during the construction stage. Differential settlement also can be accommodated by methods similar to those used to accommodate subsidence (Anon, 1975b).

Subsidence

Subsidence can be regarded as the vertical component of ground movement caused by mining operations although there also is a horizontal component. Subsidence can and does have serious effects on buildings, services and communications; can be responsible for flooding; lead to the sterilization of land; or call for extensive remedial measures or special constructional design in site development. An account of subsidence is provided in Chapter 8.

Methods of ground treatment

In recent years, there has been an increase in the extent to which the various methods of ground treatment have been used to improve subsurface conditions. Some of these techniques are not new but, in the past, they were used more as desperate remedies for dealing with unforeseen problems connected with poor ground conditions, whereas today they are recognized as part of a normally planned construction process.

Grouting refers to the process of injecting setting fluids under pressure into fissures, pores and cavities in the ground. It may either be preplanned or an emergency expedient. The process is used widely in foundation engineering in order to increase the mechanical performance or to reduce the seepage of water in the soils or rocks concerned.

If the strengthening and sealing actions are to be successful, then grout must extend a significant distance into the formation. This is achieved by injecting the grout into a special array of groutholes and is referred to as permeation grouting. Permeation grouting is the most commonly used method of grouting, in which the groutability and therefore the choice of grout is influenced by the void sizes in the ground to be treated. Normally, cement or cement–clay grouts are used in coarser soils and clay–chemical or chemical grouts are used in finer soils. The limits for penetration of particulate grouts generally are regarded as a 10:1 size factor between the D_{15} of the grout and the D_{15} size of the granular system to be injected. Generally, particulate grouts are limited to soils with pore dimensions greater than 0.2 mm (Fig. 9.30). Ordinary Portland cement will not penetrate fine sand. Because chemical grouts are non-particulate, their penetrability depends primarily on their viscosity.

Cement grout cannot enter a fissure smaller than about 0.1 mm. In fissured rocks, the D_{85} of the grout must be smaller than one-third the fissure width. There is an upper limit to this ratio as large quantities of grout have been lost from sites via open fissures. The shape of an opening also affects groutability. For the grout to achieve effective adhesion, the sides of the fissures or voids must be clean. If they are coated with clay, then they need to be washed prior to grouting. Cavities in rocks may have to be filled with bulk grouts (usually mixtures of cement, pulverized fly ash and sand; gravel may be added when large openings need filling) or foam grouts (cement grout to which a foaming agent is added).

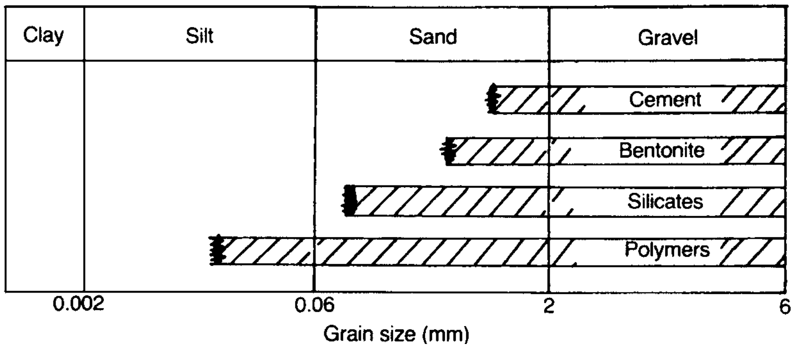


Figure 9.30

Soil particle size limitations on grout permeation.

Vibroflotation is used to improve poor ground below foundation structures. The process may reduce settlement by more than 50%, and the shearing strength of treated soils is increased substantially. Vibrations of appropriate form can eliminate intergranular friction of coarse soils, so that those initially packed loosely can be converted into a dense state. A vibroflot is used to penetrate the coarse soil and can operate efficiently below the water table. The best results have been obtained in fairly coarse sands that contain little or no silt or clay, since these reduce the effectiveness of the vibroflot.

However, today it is more usual to form columns of coarse backfill, formed at individual compaction centres, to stiffen soils. The vibroflot is used to compact these columns that, in turn, effect a reduction in settlement. Since the granular backfill replaces the soil, this process is sometimes known as vibroreplacement (Fig. 9.31). Vibroreplacement is commonly used in soft, normally consolidated compressible clays, saturated silts, and alluvial and estuarine soils. Stone columns have been formed successfully in soils with undrained cohesive strengths as low as 7 kPa. Vibrodisplacement involves the vibroflot penetrating the ground by shearing and displacing the ground around it, and then forming stone columns. It accordingly is restricted to strengthening insensitive clay soils that have sufficient cohesion to maintain a stable hole, that is, to those over 20 kPa undrained strength. These soils require treatment primarily to boost their bearing capacity, the displacement method inducing some measurable increase in the strength of the soil between the columns. Stone columns encapsulated in geofabric reinforcement may be used to transmit foundation loads below collapsible soils at the surface to suitable bearing strata beneath (Ayadat and Hanna, 2005).

Dynamic compaction brings about an improvement in the mechanical properties of a soil by the repeated application of very high intensity impacts to the surface. This is achieved by dropping a large weight, typically 10–20 tonnes, from a crawler crane from heights of

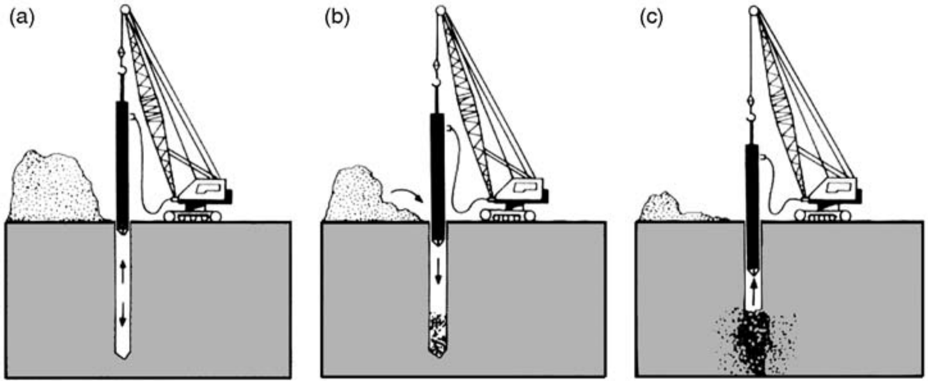


Figure 9.31

Formation of a stone column by vibrocompaction. (a) Sinking the vibrator into the soil to the depth where sufficient load-bearing capacity is encountered, (b) Aggregates are placed into the hole made by the vibrator and after each filling the vibrator is sunk again into the hole, (c) It is necessary to repeat this process as many times as may be required to achieve a degree of compaction of the surrounding soil and the aggregates as to ensure that no further penetration of the vibrator can be effected.

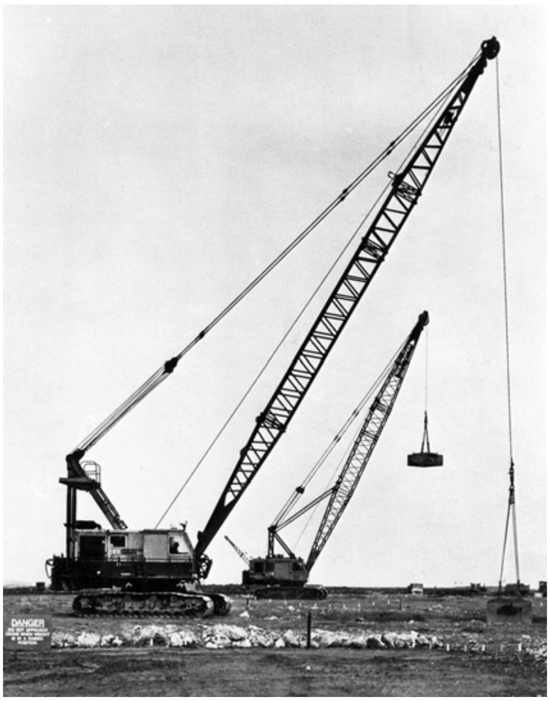


Figure 9.32

Dynamic compaction.

15–40 m at regular intervals across the surface (Fig. 9.32). Repeated passes are made over a site, although several tampings may be made at each imprint during a pass. Each imprint is backfilled after tamping. The first pass at widely spaced centres improves the bottom layer of the treatment zone, and the subsequent passes then consolidate the upper layers. In finer materials, the increased pore water pressures must be allowed to dissipate between passes, which may take several weeks. Care must be taken in establishing the treatment pattern, tamping energies and the number of passes for a particular site, and this should be accompanied by in situ testing as the work proceeds. Coarse granular fill requires more energy to overcome the possibility of bridging action, for similar depths, than finer material. Before subjecting sites that previously have been built over to dynamic compaction, underground services, cellars, etc., should be located. Old foundations should be demolished to about 1 m depth below the proposed new foundation level prior to compaction.

Lime or cement columns can be used to enhance the carrying capacity and reduce the settlement of sensitive soils. Indeed, the lime column method often can be used economically when the maximum bearing capacity of conventional piles cannot be fully mobilized (Broms, 1991). In such instances, they can be used to support a thin floor slab that carries lightweight buildings. Lime or cement columns are installed by a tool reminiscent of a giant eggbeater, the tool being screwed into the ground to the required depth. The rotation is then reversed, and lime or cement slurry is forced into the soil by compressed air from openings just above the blades of the tool. The strength and rate of increase in strength of the columns are influenced by the curing conditions.

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