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

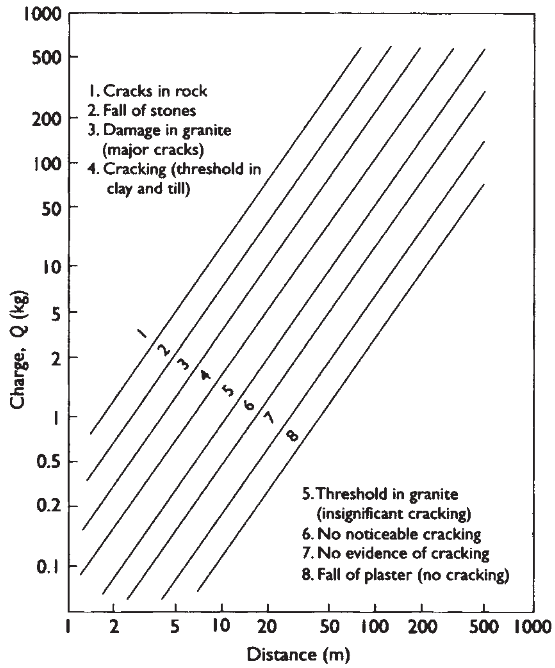

Figure 9.1

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.

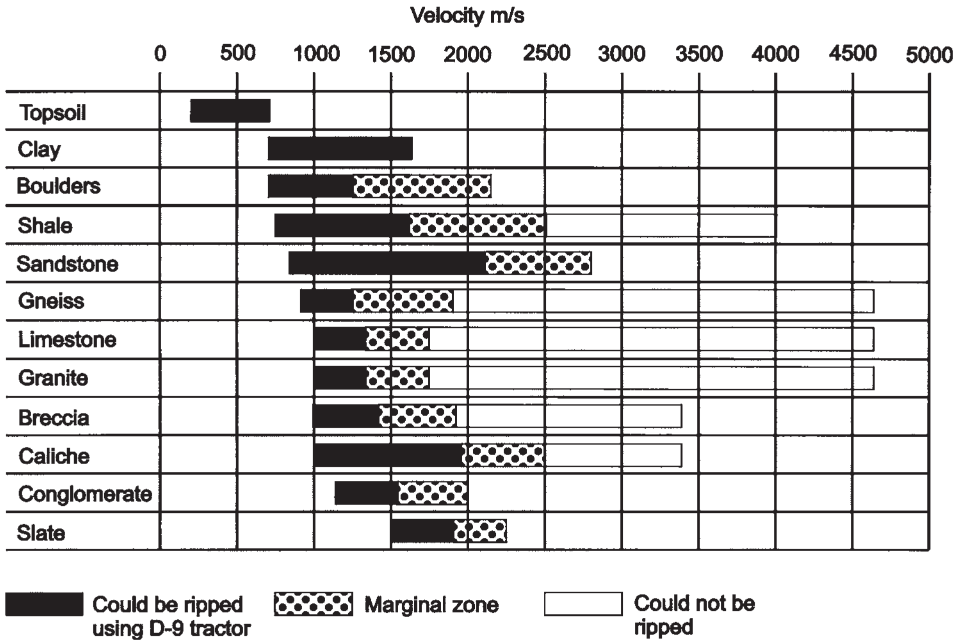

Figure 9.3

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.

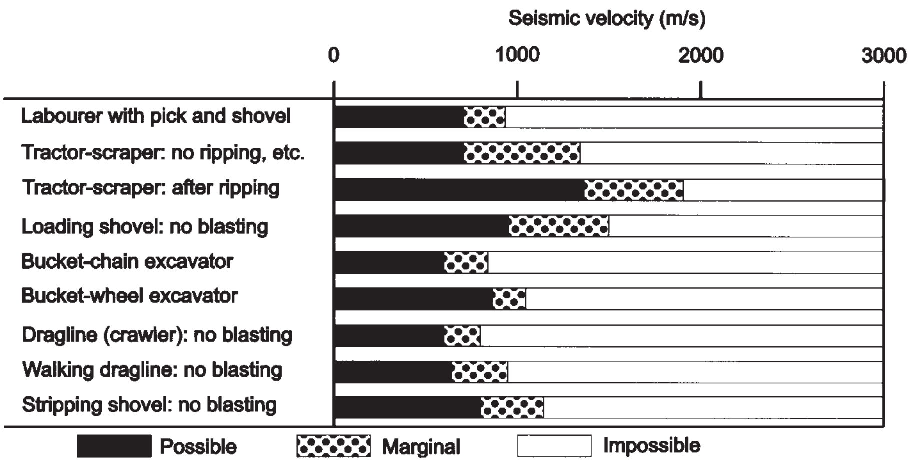

Figure 9.4

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

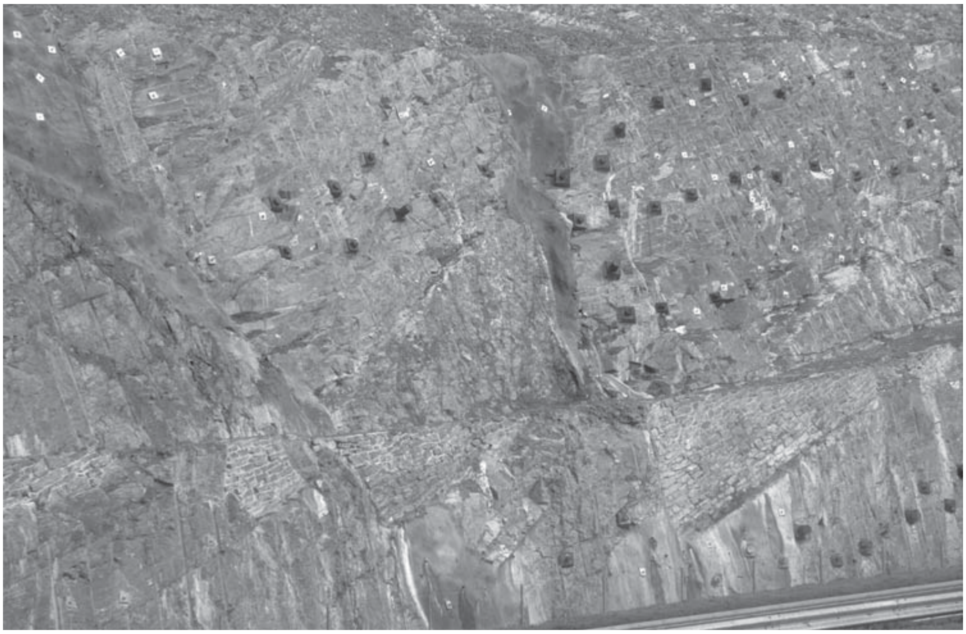

Figure 9.6

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

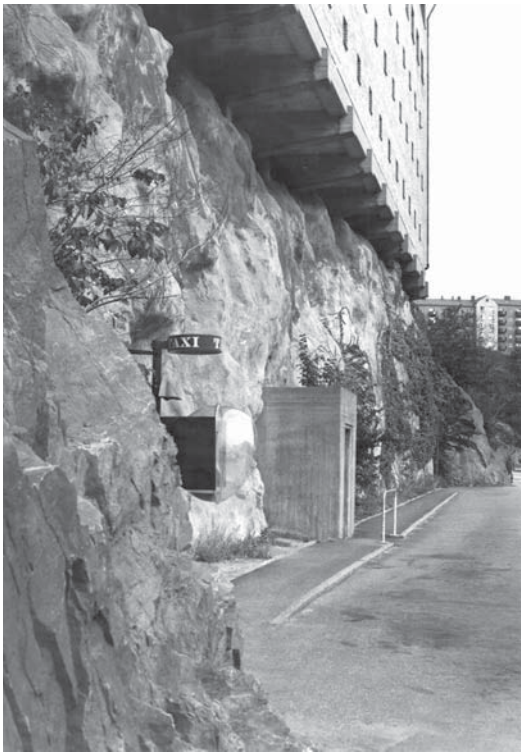

Figure 9.7

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)

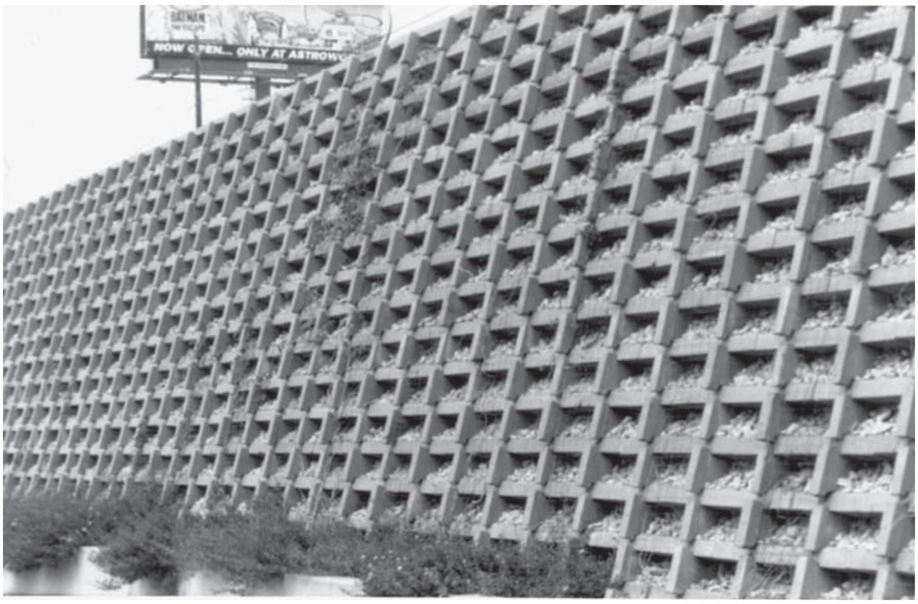

Figure 9.9

(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° Very favourable	Dip 20–45° Favourable	Dip 45–90° Fair	Dip 20–45° Unfavourable	Dip 45–90° Very unfavourable	Dip 20–45° 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.

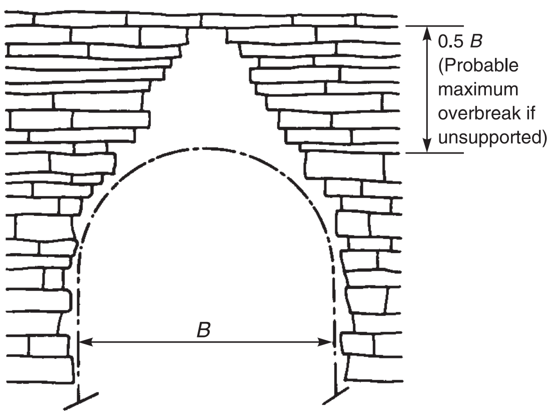

Figure 9.11

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:

$$\text{Geothermal gradient} = \frac{0.05}{k}\ (\text{approximately})\,^\circ\text{C m}^{-1} \tag{9.3}$$

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).

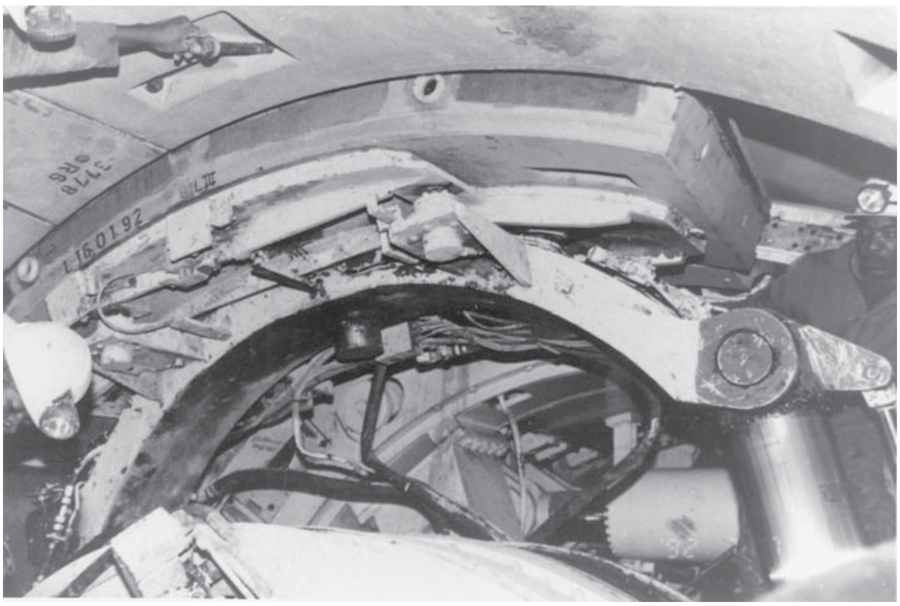

Figure 9.12

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.

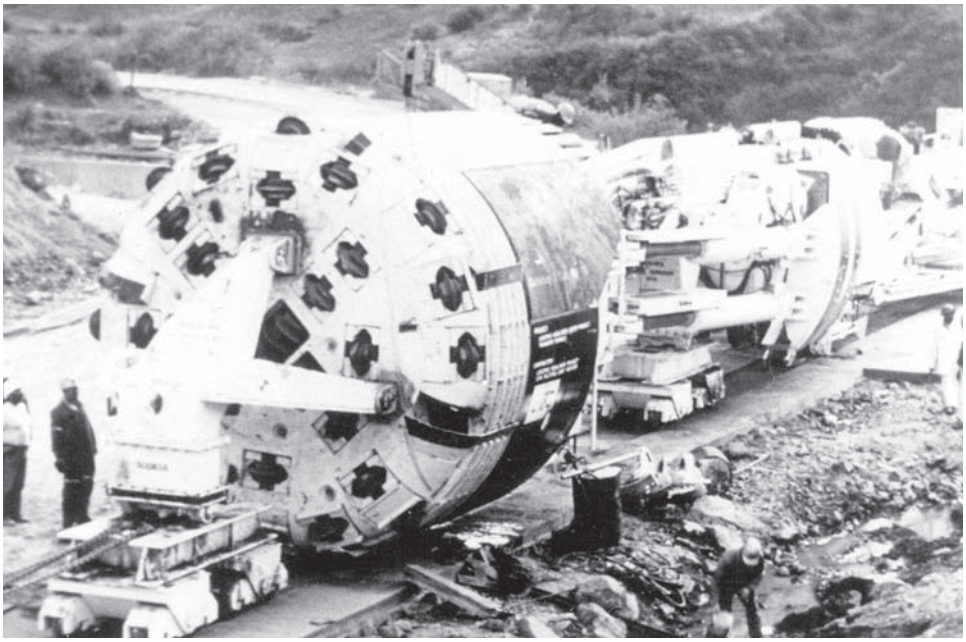

Figure 9.14

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.

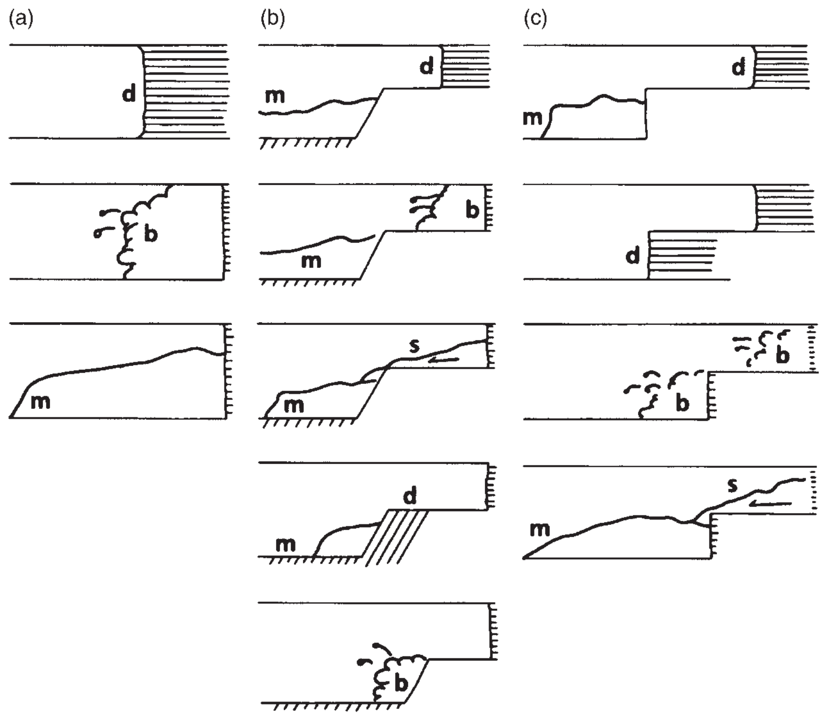

Figure 9.15

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

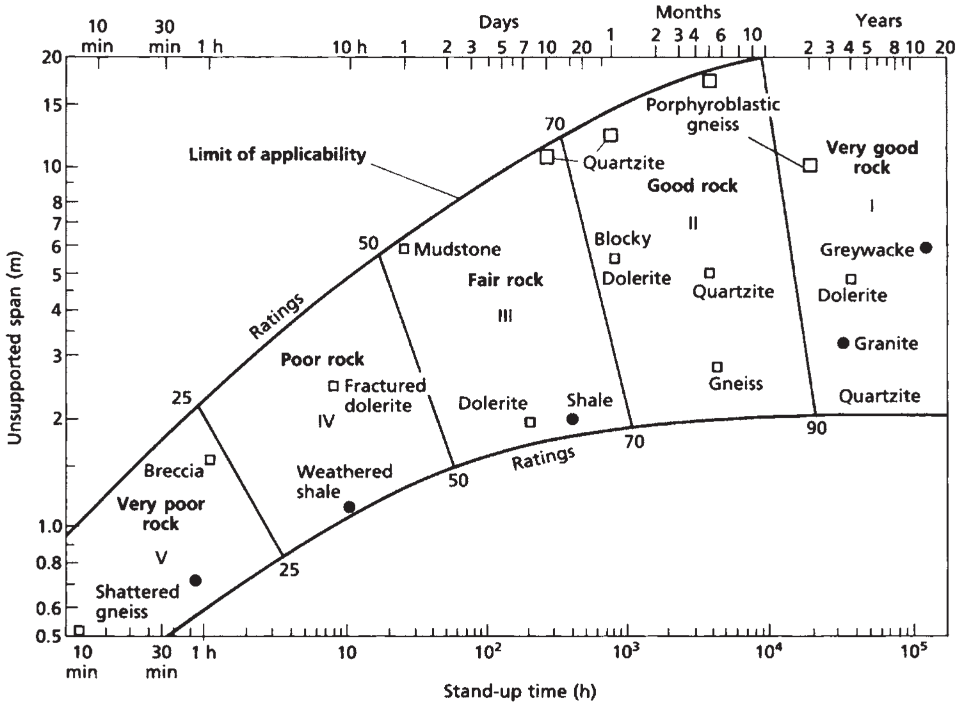

Figure 9.16

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:

$$P_i = (100 - \mathrm{RMR})\,\gamma_b B/100 \tag{9.4}$$

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		2. $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength
(b) Rock wall contact before 10 cm shear		
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

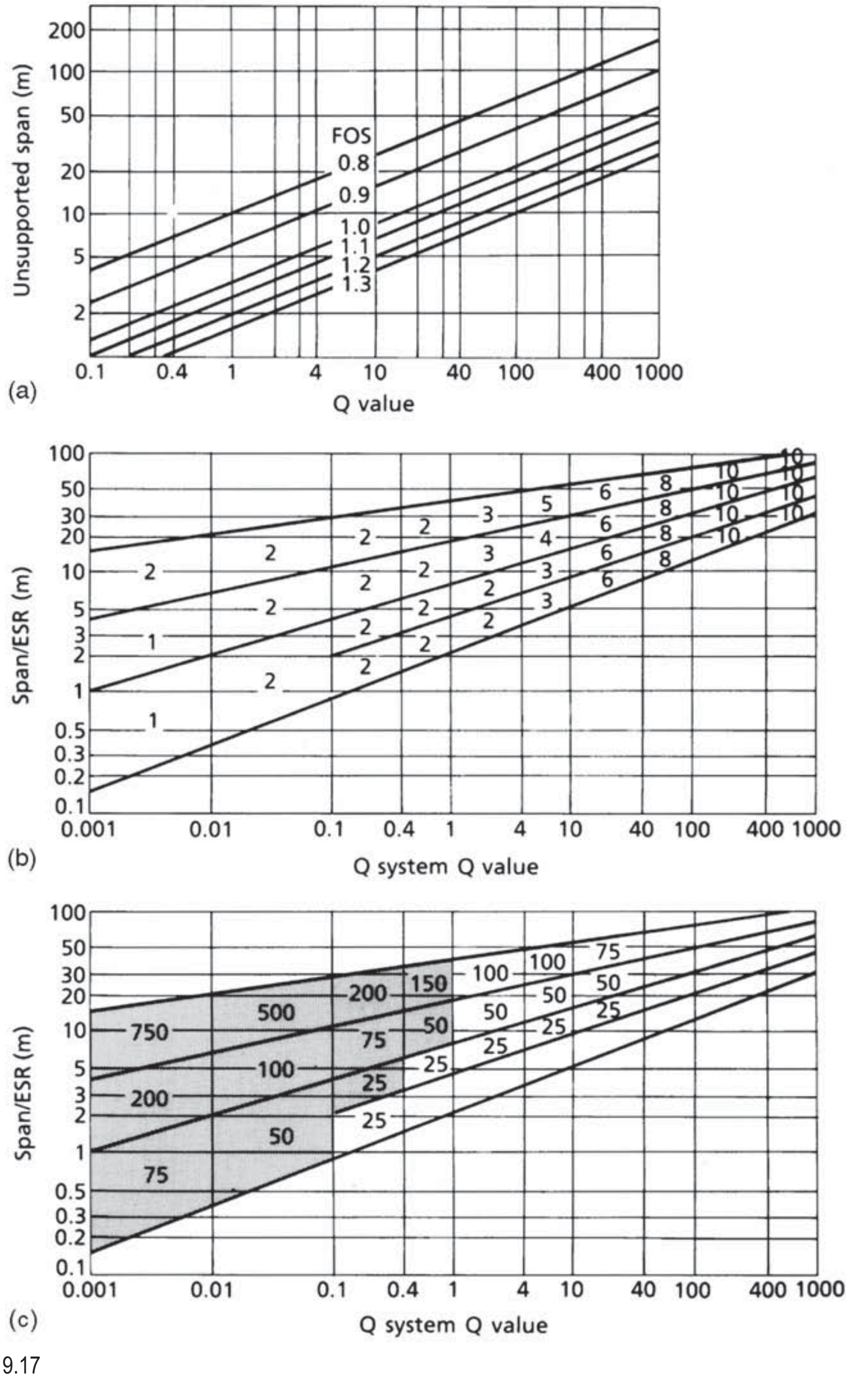

Figure 9.17

(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,

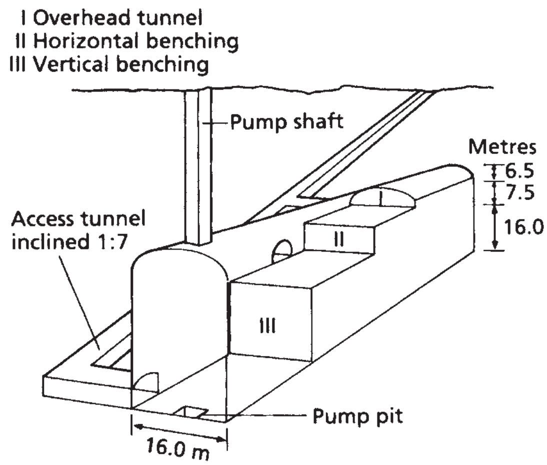

Figure 9.18

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

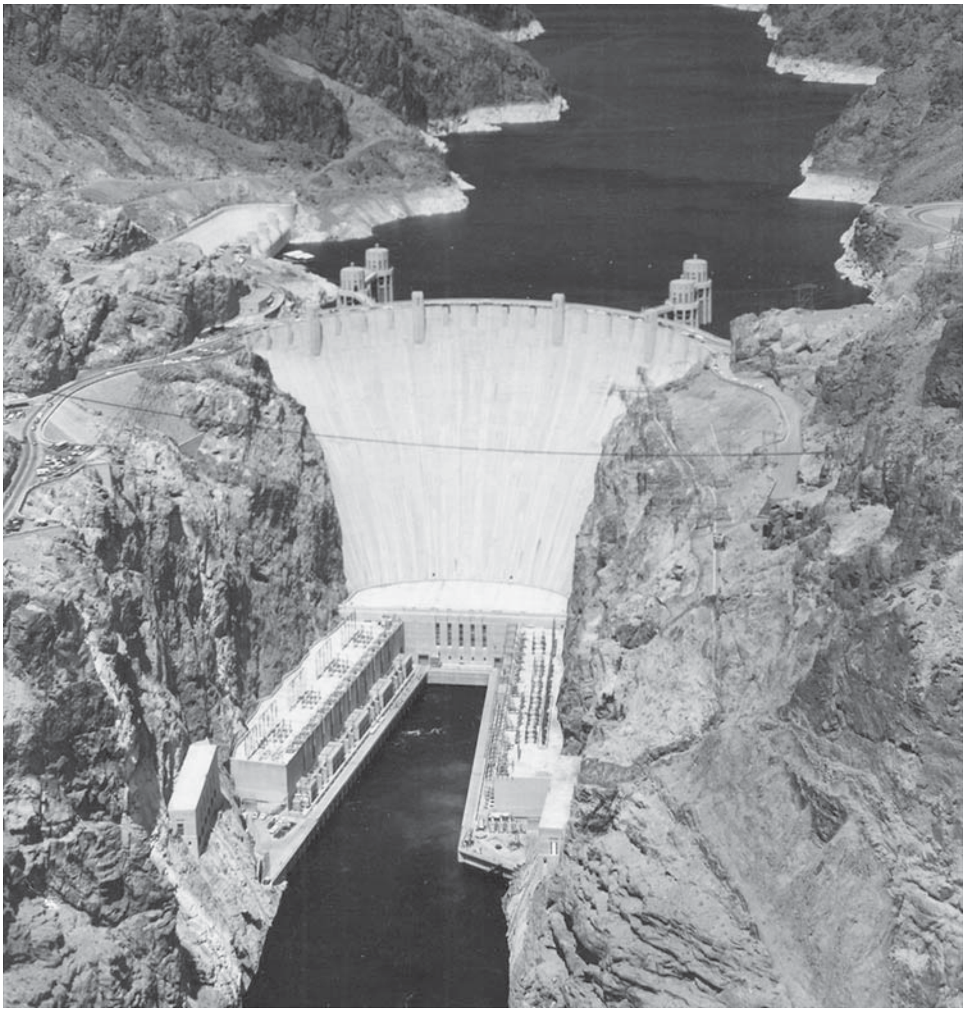

Figure 9.19

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.

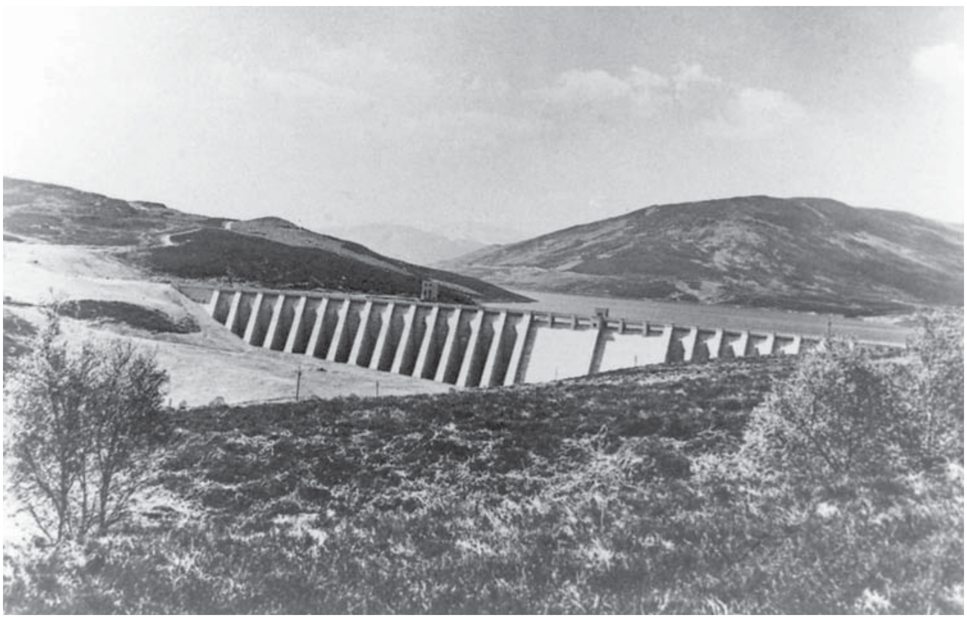

Figure 9.20

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

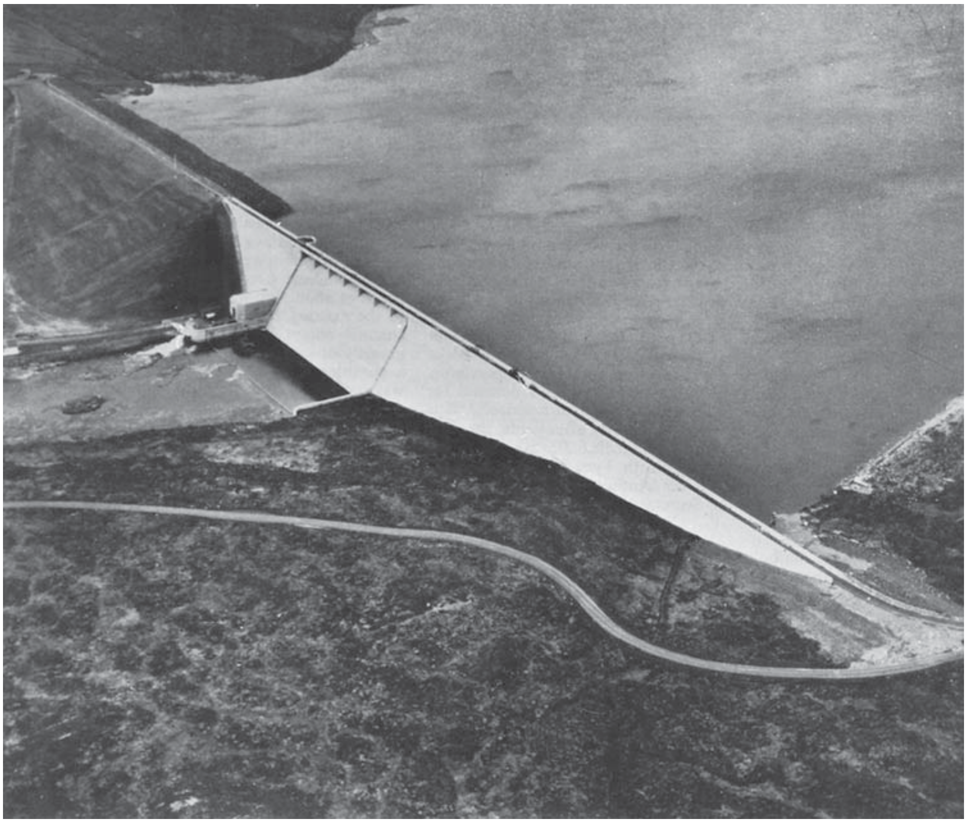

Figure 9.22

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 waterproof 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 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	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	
	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 Uniform sands; 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	

		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;

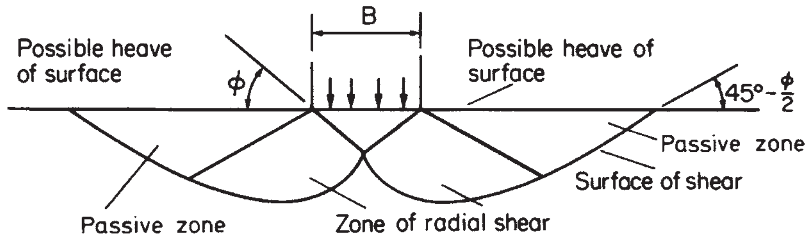

Figure 9.28

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).

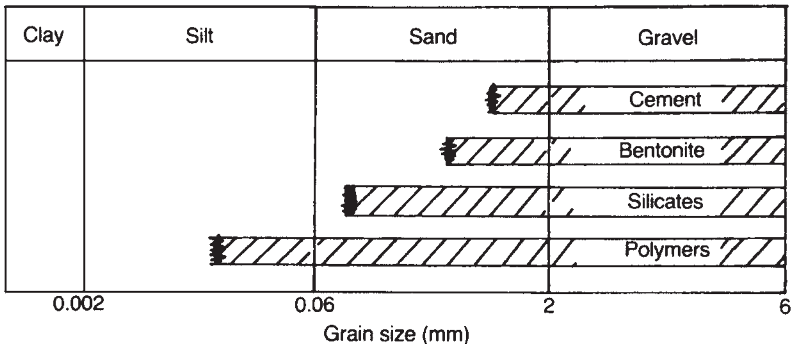

Figure 9.30

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

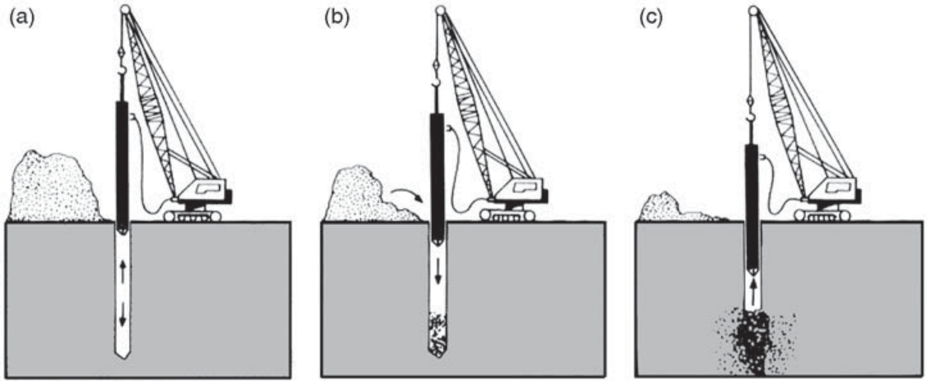

Figure 9.31

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.

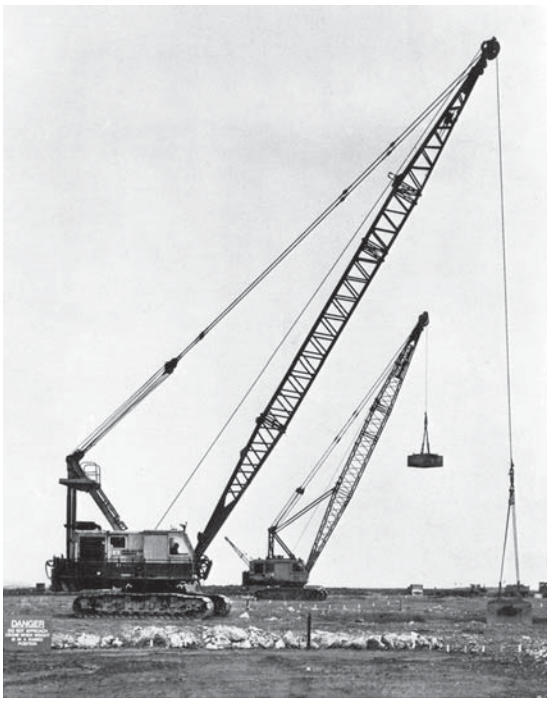

Figure 9.32

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.