The significance of engineering geology to construction

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Abstract. The paper discusses the important contribution that engineering geology makes to the construction processes of excavation and the forming of foundations. The contribution to the construction of highways is described. In particular, the influence of geology on cut slope stability, soft and hard ground excavation (including tunnelling), bearing capacity, settlement, subsidence and the choice of foundation type are all considered. Particular problems associated with the use of fills and waste materials are also mentioned. The importance of adequate site investigation is stressed.

In 1993, the International Association of Engineering Geology defined the subject of engineering geology as 'The science devoted to the investigation, study and solution of the engineering and environmental problems which may arise as the result of the interaction between geology and the works and activities of man as well as to the prediction of and the development of measures for prevention or remediation of geological hazards' (Anon 1993). This is a broad definition that covers most aspects of man's impact on the ground. However, although concern about the impact of contamination and waste disposal is now much more to the fore, the engineering geologist's contribution to civil engineering and construction projects remains at the core of the profession. This is recognized in earlier definitions of engineering geology. For example, Bell (1983) summarized the definitions as 'the application of geology to engineering practice.' More fully, this could be stated as the application of geology to civil and mining engineering, particularly as applied to the design, construction and performance aspects of engineering structures in, or on, the ground (Dearman 1972).

With respect to this core role, Knill (1975) described the main functions of an engineering geologist as providing an interpretation of ground conditions (including identification of hazards) for excavation and construction. In carrying out these functions, the engineering geologist should form part of a team concerned with the planning, investigation, design, construction and operation of engineering works. Rawlings (1972) also described the role of the engineering geologist during construction.

For any engineering geologist, and particularly those working in the civil engineering and construction industries, an understanding of the relevance of the geology to the construction process involved is essential. In the following sections, the importance of engineering geology to excavation, both at the surface and below ground, and foundations is discussed. Additionally, the importance to highways is briefly described with regard to those aspects not covered in the earlier sections.

Excavations

Excavations have been classified into two broad groups depending on whether they are made at the surface or below ground. Those at the surface are termed open excavations and they may be formed in the course of the construction of foundations, tunnels and underground chambers for a variety of purposes. Subsurface excavations include the latter two types of construction.

The geological conditions are a most important consideration in the formation of excavations affecting the method of excavation, stability of the opening and the stability of the surrounding, or overlying, ground. Attewell & Norgrove (1984) pointed out that in the recognition of the hazardous nature of undertaking subsurface excavation and the serious financial consequences of problems with this form of construction, the engineering geological investigations carried out prior to excavation are usually more extensive than for other forms of construction.

From EDDLESTON, M., WALTHALL, S., CRIPPS, J. C. & CULSHAW, M. G. (eds) 1995, *Engineering Geology of Construction.* Geological Society Engineering Geology Special Publication No. 10, pp 3–29

Slope stability

The construction of slopes arises through the formation of embankments with fill and the removal of ground to produce a cutting or a platform. In both cases the stability of the structure depends mainly on the strength of the material forming the slope or the foundation of an embankment. In turn, this may be affected by the groundwater conditions, as well as by the presence and properties of geological structures, including joints.

Various forms of failure, which may be instigated by the formation of slopes and by constructional

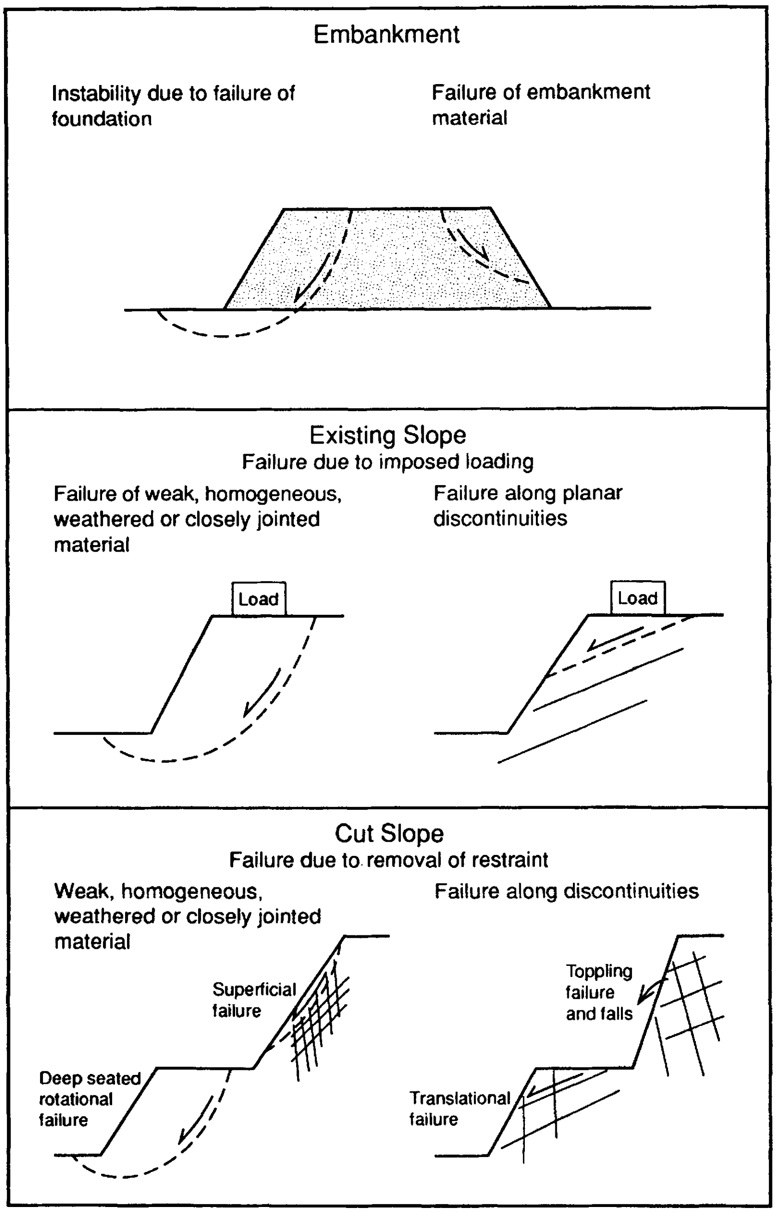

Fig. 1. Forms of slope failure due to construction.

operations for other purposes, are illustrated in Fig. 1. In soils the most common types are falls, planar sliding, rotational sliding and flows (Skempton & Hutchinson 1969). Hoek & Brown (1981) indicated that the styles of failure displayed by slopes in rocks include toppling of various types and wedge and planar failure. Rotational failure occurs, particularly, in relatively weak, weathered or closely jointed rock masses.

Many authors deal with the determination of the stability state of slopes by limit equilibrium analysis and other methods. Particularly useful guidance was provided by Hoek & Bray (1981) for rock slopes and by Bromhead (1986) for slopes in soils. Mostyn & Small (1987) and Oliphant & Horne (1992) provided a comprehensive review of limit analysis methods for soils. Such methods have the advantage that the stress–strain relationship of the material can be taken into account. The most significant causes of instability are increases in the steepness, the removal of support from the mass by excavation or erosion in the lower part of the slope, increased loading of the upper part of the slope, rises in pore water pressure within the ground and the loss of strength of the material by weathering or other process. Besides increases in pore pressure, wetting can cause failure of some slopes through the increase in weight of the material or the reduction in the magnitude of porewater suction pressures.

Slope failures during construction constitute a hazard and can be a source of delay and extra cost. In many cases they occur as a consequence of the excavation of slopes steeper and higher than will be self supporting or can be supported by any temporary works. Slopes in dry, loose granular materials will remain stable provided their inclination is less than the angle of repose of the material. Failure can be caused by the flow of water removing material by internal erosion. Such failure usually take the form of shallow translational movements. Deep-seated styles of failure may also occur in granular soils. They are usually due to the effects of the rapid lowering of ground water levels, or to excessive loading of the top of the slope.

Partially saturated granular soils may be capable of standing at high angles or even vertically for short periods of time. The length of time depends on the rate of water removal from the mass and is controlled by the density, permeability and water content of the soil, as well as the climatic conditions.

Cohesive soils are capable of standing at steep angles immediately after excavation but may fail some time later. The length of time a particular slope will remain stable depends on a number of factors. For a homogeneous and continuous soil mass, the initial, or short term, stability depends on the undrained shear strength (c_u) of the material. Rumsey & Cooper (1984) indicated that the short-term maximum stable height (H_m) of a vertical slope in the 'undrained' state can be estimated from the following relationships:

No tension crack at the head of the slope

$$H_m = 4 \times c_u/(\text{unit weight of soil})$$

Tension crack present at the head of the slope

$$H_m = 2.5 \times c_u/(\text{unit weight of soil})$$

Face supported-rotational failure extending into the base

$$H_m = 5 \times c_u/(\text{unit weight of soil})$$

Of particular importance is the presence of discontinuities, including fissures, within the soil mass, along which movements may occur. In some soil masses bedding planes or thin laminae may affect the behaviour of the material. These features not only constitute weakness surfaces within the mass, but also facilitate the movement of water through the soil.

The complex modifications to the geotechnical properties of soils that occur in response to changes in the stress and groundwater conditions are explained in soil mechanics texts, including Berry & Reid (1988). Immediately following excavation the shear strength mobilized within a homogeneous and continuous soil mass will be the undrained, or total stress value. Part of the stress imposed on the mass is borne by the pore water. However, adjustments to the new stress and water conditions entails a drop in the mobilized strength to the critical state value. The rate at which this change occurs depends on many factors including the stress history of the material and its permeability. For instance in London Clay, due to low permeability, the loss of strength of the soil mass may take some tens of years. In other cases, in particular clays containing silt laminations or extensive systems of fissures, these changes may be completed in a few hours or days.

If discontinuities are present then the shear strength mobilized within the soil mass may be less than the critical state value for the intact material. Solifluction shear surfaces and fault surfaces are weaker than other forms of discontinuity, for example joints. In practice the strength mobilized within a soil mass containing discontinuities is intermediate between the critical state value and the residual shear strength. A number of soils and fills display appreciable brittleness or capacity for strain softening. In other words, due to the process of progressive failure they gradually loose strength such that slope failure, when it takes place, occurs suddenly.

Rock masses are liable to undergo sudden and violent failure if the peak strength of the material is exceeded or an excessively steep slope is formed. The maximum heights and angles of slopes which will be stable in the short-term in rock masses can be approximately estimated from Fig. 2. The inclination and properties, particularly the angle of shearing resistance, of discontinuities within the rock mass are all important factors.

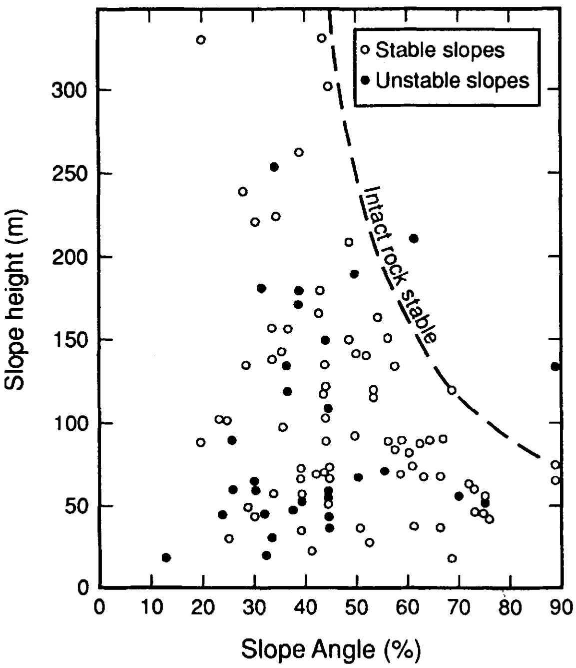

Fig. 2. Values of slope height and angle in stable and unstable rock slopes (after Hoek & Bray 1981).

The unit weight and cohesion value of the rock exert significant controls over the stability of small slopes whereas water pressures are more important in high slopes.

Depending on the conditions, rock slopes may be stable at steep angles for short periods of time but may then fail due to changes in stress and water conditions or the strain softening behaviour of joint surfaces or infilling material as described above for soil slopes. Owing to the number and complexity of the factors controlling these processes, assessing the rate at which they occur presents great difficulty. Peck (1969a) advocated continuous monitoring of rock masses as a means, by taking suitable action, of avoiding instability of this sort. Another approach is to assume that, where present, the material infilling joints or the joints themselves possess a strength no greater than the critical state value for the material. However, if the material or joints are sheared, then the strength may be as low as the residual shear strength of the material concerned.

One of the most important aspects of the excavation of slopes in rocks is the systematic collection and presentation of geological data. Matheson (1989) described methods for the collection of discontinuity data for the assessment of the stability of rock slopes. Graphical methods involving stereographic projection as described by Hoek & Bray (1981), for example, provided a rapid means of using these data in the assessment of the style of instability and stability status of rock slopes. Although very useful where full information about the discontinuities is available, a good deal of subjectivity is introduced into analyses where data obtained from rock faces or boreholes are extrapolated to give a three-dimensional view of the distribution of discontinuities within the rock mass.

Methods of excavation

The method of excavation appropriate for a particular situation depends on the ground conditions, the water conditions and other factors connected with location of the excavation itself. The equipment available for the actual process and, where required, for supporting the ground afterwards may also affect the choice.

A distinction is drawn between excavation in soft ground, in which the process may proceed without the need to loosen the ground before excavation, and that in hard ground in which drilling and blasting or mechanized excavation probably would be chosen. Problems with this construction operation frequently stem from the difficulty of predicting the ground conditions in terms of the appropriate method of excavation. For instance, the decision can hinge on very subtle differences in material properties, in weathering profiles and fault zones in which relatively unaltered material that needs to be blasted lies immediately adjacent to weaker ground that does not.

Soft ground

Ground within this category includes unconsolidated sedimentary deposits lacking cementation and highly

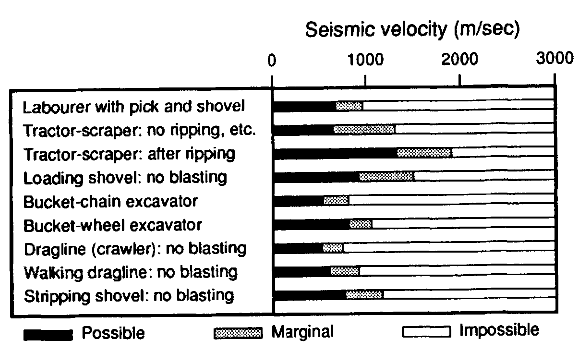

Fig. 3. Seismic velocities for determining diggability (after Atkinson 1971).

fractured or decomposed rocks that are capable of excavation by hand tools and mechanical excavators such as bulldozers and face shovels. At the stronger end of the range of materials it may be necessary to employ ripping techniques and other methods to loosen the ground before excavation.

The diggability of ground is of major importance in the selection of excavating equipment and depends primarily upon its intact strength, 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 accepted quantitative measure of diggability, assessment usually being made according to the experience of the operators and the behaviour of the ground in excavations and trial pits in the area concerned. Attempts have been made to evaluate the performance of excavating equipment in terms of seismic velocity (Fig. 3). It would appear that most earthmoving equipment operates most effectively when the seismic velocity of the ground is less than 1 km/s and will not function above 1.8 km/s.

Table 1. *Types of plant required for digging*

Machine	Distance	Material	Type of site
Bulldozer and angle-dozer	0–100 m	Most low strength soil	Small and shallow
Scraper	300 m–1 km	Not soft clay and silt	Linear and shallow
Loading shovel	Truck	Hard/dense soil; fractured rock	Shallow, small/large
Face shovel	Truck	Hard/dense soil; weak rock	Deep, small/large
Backacter	Truck	Not loose sand or soft clay	0–4 m, small
Dragline	Truck	Soft and/or flooded	Large, deep
Grabs and clam-shell bucket	Truck	Med dense sand; stiff clay	Small, deep
Bucket-wheel excavator	Truck	Dense sand, hard clay, weak rock	Large, deep

Table 2. *Excavation characteristics*

(a) In relation to rock hardness and strength

Rock hardness description	Identification criteria	Unconfined compression strength (MPa)	Seismic wave velocity (m/s)	Excavation characteristics
Very soft rock	Material crumbles under firm blows with sharp end of geological pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 3 cm thick can be broken by finger pressure	1.7–3.0	450–1200	Easy ripping
Soft rock	Can be scraped with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer	3.0–10.0	1200–1500	Hard ripping
Hard rock	Cannot be scraped with a knife; hand specimen can be broken with pick with a single firm blow; rock rings under hammer	10.0–20.0	1500–1850	Very hard ripping
Very hard rock	Hand specimen breaks with pick after more than one blow; rock rings under hammer	20.0–70.0	1850–2150	Extremely hard ripping or blasting
Extremely hard rock	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer	> 70.0	> 2150	Blasting

(b) In relation to joint spacing

Joint spacing description	Spacing of joints (mm)	Rock mass grading	Excavation characteristics
Very close	>50	Crushed/shattered	Easy ripping
Close	50–300	Fractured	Hard ripping
Moderately close	300–1000	Block/seamy	Very hard ripping
Wide	1000–3000	Massive	Extremely hard ripping and blasting
Very wide	>3000	Solid/sound	Blasting

The type of plant required for digging in a particular situation depends on the distance the excavated material needs to be transported, the nature of the material and the size and depth of the area to be excavated. Some guidelines are presented in Table 1 (Church 1981). Factors other than these may influence the choice of equipment in some cases. For instance, scrapers are most effective when employed for large earthmoving works such as road construction. A backacter may be used on small sites or those with restricted means of access. The latter can dig to close limits and make well-controlled even cuts. Draglines are especially suited to bulk excavation below track level and both these machines and bucket wheel excavators are frequently used in connection with opencast mining operations. However, smaller bucket-chain excavators can be used to dig trenches where they may achieve very high outputs in soils free from large stones. One of the advantages of large draglines is that they can pile the excavated material adjacent to the excavation when it needs to be returned as backfill. Grabs and clam-shell buckets are the most suitable type of excavator for deep excavation in confined areas such as for caissons.

In ground that is too hard for direct immediate excavation, ripping is an inexpensive method of breaking discontinuous ground or soft rock masses, before removal by earth moving machinery. The process is carried out by driving a pick into the rock mass and dragging it across the area to be excavated. The geological factors which influence rippability in rock masses include the rock type and fabric, intact strength and degree of weathering, rock hardness and abrasiveness, and the nature, incidence and geometry of discontinuities (Table 2). The latter reduce the overall strength of the rock mass, their spacing governing the amount of this reduction. The greater the amount of gouge or soft fill along discontinuities, the easier it is to excavate by ripping. Strong massive and hard abrasive rocks do not lend themselves to ripping. On the other hand, sedimentary rocks such as well-bedded and jointed sandstone and limestone or thinly interbedded strong and weak rocks may be amenable to ripping rather than blasting. Generally speaking, coarse-grained rocks are easier to rip than fine-grained ones.

According to Atkinson (1970) the most common method for determining whether a rock mass is capable of being ripped is seismic refraction using a correlation such as that shown in Fig. 4. The practical limit for ripping is a seismic velocity of about 2 km/s. However, loosening the ground by light blasting or hydraulic fracturing may render stronger ground suitable for ripping. Kirsten (1982, 1988) cautioned against an over-dependence on seismic velocity as a criterion for ripping suitability, particularly since it cannot be determined to an accuracy better than about 20%.

Hard ground

Ground which is too strong and massive to be excavated by digging or ripping will need to be loosened and fragmented by blasting before excavation. Explosive charges are detonated in holes drilled into the rock mass. There are circumstances, for example if the vibrations, noise and dust produced in course of blasting are a problem, in which another means of excavation may need to be used.

The success of carrying out excavation by blasting in a particular rock mass depends on matching the properties of the rock mass in terms of strength and the discontinuities present to the spacing, depth and orientation of the blast holes and the size of the explosive charges used. In an optimum blasting operation the fragmented rock will have the characteristics required for any subsequent operations, including mechanized excavation and after-use.

The rate of blasting depends on the rate at which holes can be drilled. According to McGregor (1967) the properties of a rock mass which influence drillability include strength, hardness, toughness, abrasiveness, grain size and the openness and properties of discontinuities. Obviously difficulties with drilling are liable to arise if the properties of the rock mass are more adverse than anticipated, so care needs to be taken in assessing rock masses which are liable to show variation in any of these features. For example the size of the fragments produced during drilling operations influences the rate

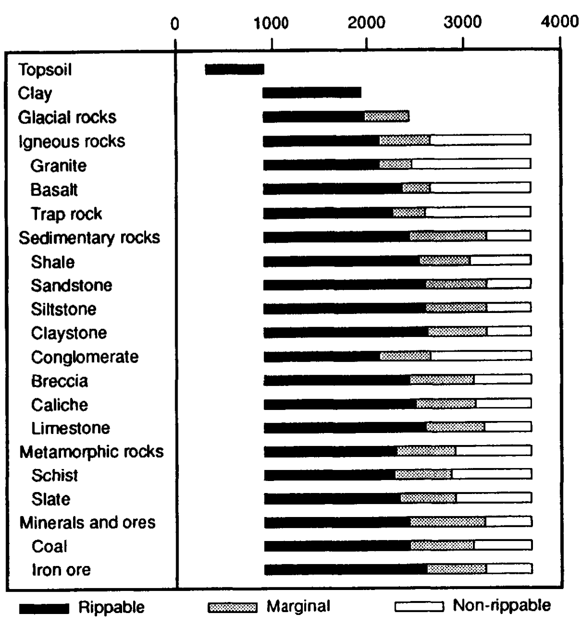

Fig. 4. Seismic velocities in relation to rippability.

Table 3. *Types of explosives*

Explosive	Rock strength	Water resistance	Bulk strength	Density
Ammonium nitrate-fuel oil (ANFO)	Low	Poor	Low	Low
Slurry immusion	Low	Poor		
Gelignite	High	Good	High	High
High density dynamite	High–soft	Fair–medium	High	
Ammon dynamites	Low–medium	Medium	Medium	
Belex	Low–high	Good	High–low	High–low
Trimonite		Poor–good		Medium
Slurry		Good		

of drill bit wear. Although large fragments may cause scratching of a bit they bring about comparatively little wear, whereas the production of dust in tougher but less abrasive rock causes polishing. This may lead to the development of high skin hardness on tungsten carbide bits which in turn may cause them to spall. Even drilling bits with diamond insets lose their cutting ability upon polishing. Rock fabric determines the characteristics of the chippings produced and most coarse grained rocks can be drilled more quickly than can fine-grained varieties or those in which the grain size is variable.

The ease of drilling is also affected by the presence and orientation of discontinuities in the rock mass. Depending upon the orientation and character of discontinuities, the holes may be deflected off line, the bit may experience excessive friction, or there may be a partial loss of the drilling energy controlling penetration.

Holes of up to 50 mm diameter can be sunk in rocks of soft to medium hardness, such as some sandstones, shale, coal, gypsum and rock salt, with rotary drills. Larger holes up to 100 mm diameter, can be made in soft rocks by reaming the original hole. Stronger rocks require the use of rotary percussion drills which combine a hammering action with bit rotation. This process is most effective in brittle materials since it relies upon chipping the rock. Care needs to be taken with the drilling of the blast hole, particularly those forming the front row of a blast area. Overbreak in holes and the redrilling of holes is a particular problem in some situations.

The presence, or otherwise, of water, the rock strength and the degree of fragmentation compared with the existing discontinuities all have an important influence on the choice of explosive material. Some types of explosive material are listed in Table 3. Although slurry and ANFO explosives can be placed in plastic sleeves or holes can be pumped dry before charging, in practice there is a danger of incomplete fragmentation of rock masses due to water dilution. However, these explosives are relatively cheap and safe to use. Also, these low-density explosives are useful where excessive fragmentation of the rock needs to be avoided. Higher bulk density explosives enable the holes to be more widely spaced but cartridge types may cause excessive fragmentation at the sites of the actual charges. The powder explosives have low densities and are of relatively high weight strength. They are especially suitable for use in soft to medium strength rocks which are moderately dry (Anon 1972). Blasting agents such as Nobelite and Amobel can be used in open-cast mining and quarrying where large diameter drilling is employed for overburden blasting.

Rock breakage in blasting is controlled by the character of the rock itself, especially the strength and degree of fracturing, as well as the relation between the burden and the hole spacing and the timing of detonation in adjacent holes. Short delays yield maximum rock fragmentation but minimize ground vibrations.

Approximate quantities of explosive for different rock situations are suggested in Table 4. Generally, 1 kg of high explosive will bring down about 8 to 12 tonnes of rock. In soft and highly fractured rock masses, including some mica schists, there is a danger that, due to a lack of transmission of the explosive energy, the rock immediately surrounding the blast hole is pulverized and the area between the holes is not fractured. The early movement of the rock face is most important for an efficient blast in which good fragmentation per drill hole is achieved with minimum quantity of explosive.

In very hard rock an explosive with high energy and concentration is required but in medium strength rocks a high velocity detonation produces a shattering effect. A medium to high explosive should be used in medium to hard laminated rocks and the greatest efficiency is obtained with fairly bulky explosive in soft to medium rocks. Table 5 indicates the approximate amounts of charge in relation to burdens for primary blasting.

Table 4. *Weight of explosive charge in a single row of holes*

Ground conditions	Weight of explosive per cubic metre of rock excavated (kg)
Soft laminated strata	0.15 to 0.25
Hard sedimentary strata	0.45
Jointed igneous rocks	0.6

Table 5. *Typical charges and burdens for primary blasting by shot-hole methods (adapted from Sinclair 1969)*

Minimum finishing diameter of hole (mm)	Cartridge diameter (mm)	Depth of hole (m)	Burden (m)	Spacing (m)	Explosive charge (kg)	Rock yield (tonnes)	Blasting ratio	Tonnes of rock per metre of drillhole
25	22	1.5	0.9	0.9	0.3	3	10.0	2.0
35	32	3.0	1.5	1.5	1.8	19	10.5	6.3
57	50	6.1	2.4	2.4	9.5	97	10.2	15.9
75	64	9.1	2.7	2.7	18.0	180	10.0	19.8
75	64	12.2	2.7	2.7	25.0	245	9.8	20.1
75	64	18.3	2.7	2.7	36.3	365	10.1	20.0
100	83	12.2	3.7	3.7	43.0	430	10.0	35.3
100	83	30.5	3.7	3.7	104.3	1120	10.7	36.7
170	150	18.3	6.7	6.7	216.5	2235	10.3	122.1
170	150	30.5	6.7	6.7	363.0	3660	10.1	120.0
230	200	21.3	7.6	7.6	329.0	3350	10.2	157.3
230	200	30.5	7.6	7.6	476.0	4670	9.8	153.1

Although formulae for predicting the relationships between yield and hole spacings exist (Roberts 1981; Vutukuri & Bhandari 1961), careful trials provide the best means of determining the burden and blasting pattern in any particular rock.

Blasting of two or more dissimilar rock masses frequently occurs in open excavation. Where it is impractical to select a different blasthole diameter for each rock mass a compromise value has to be chosen. Inclined blastholes are more difficult to drill, but are effective in eliminating excessive front-row toe burdens when dealing with rock masses with a sloping free face. They also yield good fragmentation and have the potential for producing smoother, more stable faces.

In many excavations it is important to keep overbreak to a minimum. Apart from the cost of removing extra material which then has to be replaced, damage to rock forming the walls or floor may lower the bearing capacity and necessitate further excavation. Usually, smooth faces are more stable, especially in the long-term. The *in situ* dynamic tensile breaking strain and, more particularly, the nature, frequency, orientation and continuity of discontinuities control the extent of overbreak. The amount of overbreak can be limited by reducing the density of the explosive or by decoupling and/or decking back-row charges (Hagan 1979). Starfield (1966) indicted that centrally-initiated charges minimize both overbreak and ground vibrations.

Fissile rocks such as slates, phyllites and schists tend to split along the planes of cleavage or schistosity if these run at a low angle to the required face. This may be controlled by the use of closely spaced lightly-charged or empty line holes between the blast holes (Paine *et al.* 1961). Pre-splitting techniques, in which a free surface or shear plane is formed in the rock mass prior to the detonation of the main charges, involve the controlled usage of explosives in approximately aligned and evenly spaced small diameter drillholes at the perimeter of the required excavation. The pre-split surface then acts as a limiting plane for the blast proper. The detailed procedure is based usually on the results of trials.

In many urban areas consideration must be given to the transmission of vibrations to near-by structures. The shock waves are attenuated with distance from the blast, the higher frequencies being maintained more readily in dense rock masses. Rapid attenuation occurs in unconsolidated deposits which are characterized by lower frequencies. If the use of explosives is prohibited then alternative methods of rock breakage, such as hand-held pneumatic or lorry-mounted diesel-powdered breakers, may be used. Unfortunately, both of these methods are noisy and cause vibrations, so hydraulic bursters have been used as an alternative for producing large excavations. In this latter technique a line of holes is formed by a rotary diamond tipped core drill or oxygen lance. The rock is then split by enlarging hydraulic rams inserted into the holes. With holes about 150 mm diameter, inserted to a depth of 2 to 3 m, the burden and spacing of holes should be about 1 m. Obviously, use is made of discontinuities, especially bedding planes, when this method is employed.

Tunnels

The construction of tunnels requires a considerable geological input. Constructional problems include the maintenance of the stability of the excavations, surface subsidence and difficulties with the excavation operations. The investigation of the ground prior to construction is complicated by the linearity of tunnels. This is important since only a small deviations from the predicted geological conditions can result in delay and additional cost because the design of the structures and the methods of excavation and support will not be the optimum ones. In areas of complex geology it may prove impossible to obtain an unambiguous interpretation of the geological conditions. Even in relatively simple areas, a reliable geological interpretation may require a large number of deep boreholes to be drilled.

It is important that an attempt is made to predict the ground conditions along the entire tunnel alignment. In bedded strata investigated by drilling boreholes, the holes should be sufficiently close together and deep enough for the confident correlation of recognizable beds between the holes. It is also important to obtain samples of the ground at the tunnelling horizon. In areas of dipping strata it is advisable to drill holes on a triangular pattern rather than just along the tunnel alignment. This enables the dip and strike of the strata to be determined and can assist the determination of the locations of folds and faults.

In tunnels in superficial or residual materials above bedrock care should be exercised to ensure that the position and nature of the rockhead is known. Many problems arise with tunnels inadvertently positioned such that rockhead occurs within the excavations. On the other hand, for some tunnels expected to be excavated entirely within the bedrock, the presence of a buried valley or solution feature has been the source of serious difficulties. This involves identifying the engineering rockhead (where the material has rock properties), as well as the geological rockhead.

Old mine workings, whether above, at, or below the tunnelling horizon, need special attention. Those above the tunnel may influence the amount of subsidence experienced at the ground surface. However, more serious for many tunnels below the groundwater table is the possibility of flooding of the excavations. If the workings are close to the tunnelling horizon they may also have an adverse effect on the stability of the excavations. On the other hand, the subsidence of old workings below the tunnel would give rise to unexpected *in situ* ground stresses and also prejudice the long-term stability of the works.

Faults and zones of disturbed ground may give rise to unpredicted difficulties, particularly with respect to stability and water ingress. The locations and nature of such features are usually difficult to predict during pre-construction investigations but probes drilled into the face ahead of the excavation can provide forewarning. The character of the discontinuity surfaces frequently has a significant influence on the stability of rock masses, for example, shear zones and slickensided surfaces often give rise to unstable conditions.

Difficulties may also arise because of the presence of materials with unexpected, adverse properties. For instance, some rocks readily disintegrate during excavation and if the conditions are wet the resulting slurry can be difficult to remove from the excavations. The presence of a relatively thin band of very hard material, especially if situated at the tunnel invert, can lead to significant problems with excavation. Extra cost arises from the reduction in the rate of driving as well as the need to modify the design and construction procedures. Generally speaking the position is more serious if the adverse feature runs with the tunnel alignment, rather than crossing it at a steep angle.

Machine tunnelling in hard rock

Mechanical cutting using a tunnel boring machine (TBM) or blasting is used in hard rock tunnelling. The performance of tunnel boring machines is more sensitive to changes in rock properties than conventional drilling and blasting techniques, consequently their use in rock masses which have not been thoroughly investigated involves a high risk.

Apart from ground stability and support, the most important economic factors in machine tunnelling in hard rock are cutter wear rate and penetration rate. The rate of cutter wear is affected by the condition of the rock mass, such as whether it is massive, jointed, weathered or folded, and the abrasiveness of the rock material. Abrasiveness is, itself, a function of quartz content, grain size and the strength and porosity of the rock. Pirie (1972) suggested that cutting costs generally become prohibitive if the uniaxial compressive strength of the rock is above about 140 MPa.

Tunnelling boring machines have been used successfully to excavate many rock types, generally at greater speed than conventional methods (Duddeck 1992). They are particularly suited to excavating rocks with uniaxial compressive strengths between 70 and 150 MPa. The stresses imposed on the surrounding rock by machine tunnelling are much less than those produced during blasting. Consequently smooth tunnel walls, with little overbreak and the reduced need for ground support, are the likely outcome.

Drilling and blasting

The conventional method of advancing a tunnel in hard rock is by drilling and blasting techniques. In good ground, a full-face can be fired simultaneously but where variable or poor ground is present the top heading and bench or similar method might be more suitable. It is a basic principle of tunnel blasting that a cut should be opened up approximately in the centre of the face in order to provide a cavity into which subsequent shots can blast. Delay detonators allow a full-face to be charged, stemmed and fired, the shots being detonated in a predetermined sequence.

Depending on the properties of the rock mass and the blasting technique drilling and blasting can damage the rock mass. This can increase the need for ground support and may increase water ingress into the excavations. The disturbance due to blasting can also compromise the stability of the excavations. Overbreak, in particular, can be reduced by accurate drilling and careful blasting. Controlled blasting may be achieved either by pre-splitting the face to the desired contour or by smooth blasting techniques.

The stability of the opening depends on establishing a new state of equilibrium in the material around the excavation, and a new state of equilibrium is established in which shearing stresses give rise to arching around the opening. Massive igneous rocks and horizontal or gently dipping sedimentary rocks which strike parallel to the tunnel axis and steeply dipping formations where the strike is normal to the tunnel axis generally offer favourable arching possibilities. However, in badly fractured rocks and weak ground, arching patterns tend to be poorly developed and additional support needs to be provided. These local changes to the stress regime can lead to various types of failure of the ground around the opening. Figure 5 shows the possible fracture sequence for tunnels subject to increasing levels of vertical pressure.

In certain areas, particularly orogenic belts, the state of stress is influenced not only by the overburden but also by residual and active tectonic stresses. The removal

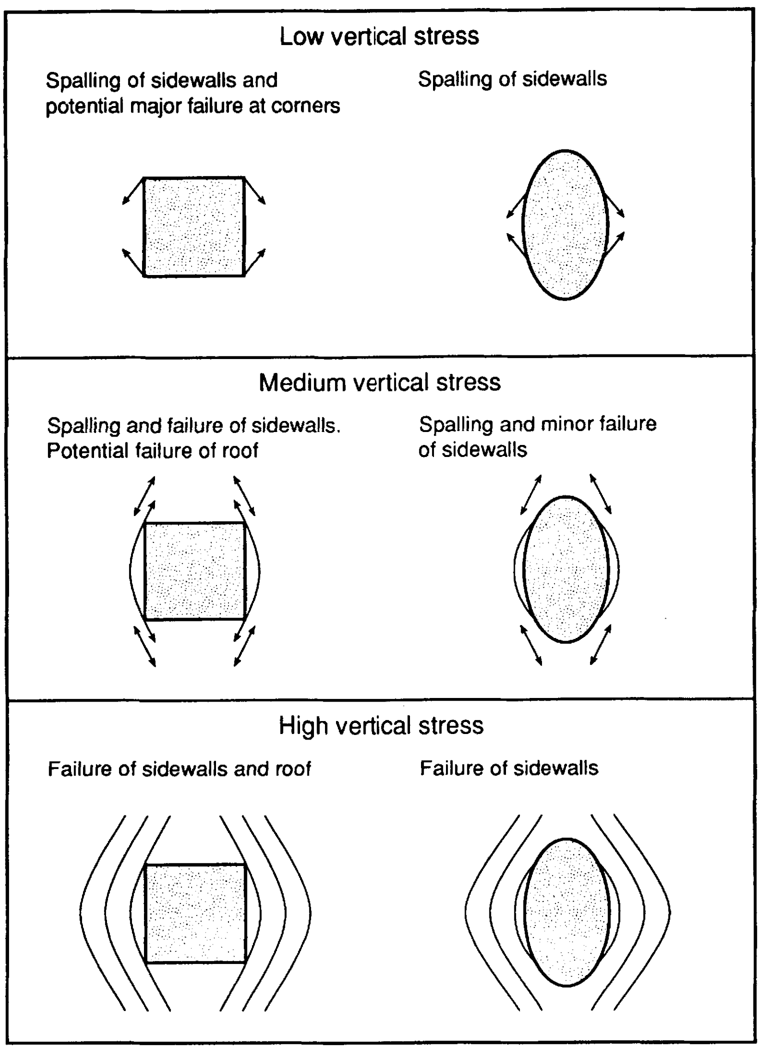

Fig. 5. Effect of vertical stress magnitude on failure in square and elliptical tunnels in hard quartzite (after Hoek 1966).

of the confining pressure during tunnelling can lead to a effects ranging from gradual deformation and fracturing of the rock mass to sudden rock bursting.

Instability and support

The time a rock mass may remain unsupported in a tunnel mainly depends on the magnitude of the stresses within the unsupported rock mass. Thus the bridging capacity or stand-up time of a particular rock mass is a function of the width of span and the strength, discontinuity pattern present as well as the *in situ* stress regime of the ground. Rock pressures on the lining of a tunnel are influenced by the size and shape of the tunnel with respect to the intact strength of the rock mass concerned and the nature of the discontinuities, geostatic stress regime, the groundwater pressures, method of excavation, degree of overbreak, the length of time before placing the permanent lining and the stiffness of the lining system.

The joint pattern often proves one of the most difficult and crucial factors to appraise when determining the type of support system to employ. In addition to defining the dimensions and orientation of the joint pattern it is necessary to evaluate the conditions of the joint surfaces and the effects of the tunnel size, direction of drive and method of excavation.

Important features of the joints include the amount of separation between adjacent faces, their continuity and roughness, as well as the presence and type of any infill material. While tight joints with rough surfaces and no infill material have a high strength, open continuous ones facilitate block movement and high groundwater flows. The continuity of the joints in a particular direction influences the extent to which the rock material and the joints separately affect the behaviour of the rock mass.

Where necessary, primary support for a tunnel in rock may be provided by rock bolts, shotcrete, arches or the installation of the permanent lining at the time of excavation. Rock bolts can be used to secure individual blocks of rock or to strengthen the rock mass and improve its bridging capacity. Shotcrete can be used for lining tunnels, for example by filling discontinuities and binding together adjacent rock surfaces, by providing resistance against rock falls and, when applied in a thick layer (150 to 250 mm) by giving structural support (Farmer 1992). Shotcrete can be combined with rock bolting and steel mesh and, in very unstable rock masses, steel arches can be added to provide additional support. As discussed below, the design of the support system is usually based upon a geomechanical classification of the rock mass.

Classification of rock for support

Rock mass classification provides a means of assessing the most appropriate method of excavation in terms of the rock mass concerned and of designing the type of tunnel support. Such a classification needs to be based

Table 6. *Guide for selection of primary support in tunnels at shallow depth, size: 5 to 15 m; construction by drilling and blasting (after Bieniawski, 1974)*

Rock mass class	Alternative support systems		
	Mainly Rockbolts (20 mm dia., length $\frac{1}{2}$ tunnel width, resin bonded)	Mainly Shotcrete	Mainly Steel ribs
I		Generally no support is required	
II	Rockbolts spaced at 1.5 to 2.0 m plus occasional wire mesh in crown	Shotcrete 50 mm in crown	Uneconomic
III	Rockbolts spaced at 1.0 to 1.5 m plus wire mesh and 30 mm shotcrete in crown where required	Shotcrete 100 mm in crown and 50 mm in sides plus occasional wire mesh and rockbolts where required	Light sets spaced at 1.5 to 2 m
IV	Rockbolts spaced at 0.5 to 1.0 m plus wire mesh and 30 to 50 mm shotcrete in crown and sides	Shotcrete 150 mm in crown and 100 mm in sides plus wire mesh and rockbolts, 3 m long spaced 1.5 m	Medium sets spaced at 0.7 to 1.5 m plus 50 mm shotcrete in crown and sides
V	Not recommended	Shotcrete 200 mm in crown and 150 mm in sides plus wire mesh, rockbolts 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

Table 7. *The Rock Mass Rating (geomechanics classification of rock masses) (after Bieniawski 1989)*

(a) Classification parameters and their ratings

	Parameter		Ranges 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 strength 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 continuous No separation Unweathered wall rock	Slightly rough surfaces Separation <1 mm Slightly weathered walls	Slightly rough surfaces Separation <1 mm Highly weathered wall	Slickensided surfaces or Gouge <5 mm thick or separation 1–5 mm continuous	Soft gouge >5 mm thick Separation >5 mm Continuous
	Rating		30	25	20	10	0
5	Groundwater	Inflow per 10 m tunnel Length (l/min)	None	<10	10–25	25–125	>125
			or	or	or	or	or
	Ratio	$\frac{\text{Joint Water pressure}}{\text{Major principal stress}}$	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

(b) Rating adjustment for discontinuity orientations

Strike and dip of discontinuities		Very favourable	Favourable	Fair	Unfavourable	Very favourable
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.		1	11	111
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	3 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

on fundamental properties of the particular rock mass and it should be applicable both to drill hole core and direct observation of the rock mass. Lauffer's (1958) classification related the unsupported rock-span to the stand-up time. Wickham *et al.* (1972) advanced the concept of rock structure rating (RSR) into rock mass classification and related the quality of rock to ground support in tunnels. For this, the relative effects on the requirements for ground support are rated in terms of rock structure, the joint pattern and groundwater inflow.

An alternative system of rock mass classification was introduced by Bieniawski (1974, 1989). Table 6 indicates the primary support measures for shallow tunnels 5 to 15 m in diameter, driven by drilling and blasting. In this system, a 'Rock Mass Rating' (RMR) is derived taking into account a number of rock mass and material parameters. The system is summarized in Table 7. Two classifications systems have been developed in South Africa (Kirsten 1982, 1988) for the excavation of rock masses, one based on the geomechanics system of Bieniawski (1974) and the other on the Q system of Barton *et al.* (1975). Barton *et al.* (1975) also introduced a sophisticated rating system for this purpose in which the characteristics of the joints were highlighted. Particular adverse conditions including the effects of squeezing and swelling ground were also taken into account.

Rock mass quality (Q), together with the support pressure and the dimensions and purpose of the underground excavation are used to estimate the type of suitable permanent support. The Q value is related to the type and amount of support by deriving the equivalent dimensions of the excavation. The support values suggested in the charts are for primary support. The values should be doubled for a permanent lining. Although these methods are useful in the design of tunnel support systems, problems may arise. For example, the effects of blast damage, *in situ* stress regime and rock durability are not taken into account.

Influence of joints and faults

The effect of joint orientation in relation to the tunnel axis is given in Table 8. Unexpected instability is liable to occur if the orientation and character of the discontinuities lie outside the expected limits. Particular problems may arise due to overbreak leading to the excess handling and need for backfilling but instability may also constitute a hazard during construction and compromise the stability of the structure and ground surface. The amount of overbreak is influenced by the character of the rock type and its discontinuity pattern, as well as the method of excavation, the length of unsupported tunnel and the time lapse between excavation and the provision of support.

In rock masses in which the joint spacing is wider than the width a tunnel, then the beds bridge the tunnel as a solid slab but if the bedding is relatively thin roof failure by bending may occur leading ultimately to overbreak into an arch form. The roughness, and other features of the discontinuity surfaces, and the presence and type of joint in-fill have a significant effect on the behaviour of rock masses. For instance clay gouge may seal the joints against the ingress of water. However, such material may result in the slip of blocks along unfavourably orientated joints into the excavation.

Zones of disturbed ground and non-uniform rock pressures on a tunnel lining associated with faults are a common cause of problems during the construction of tunnels. Generally, problems increase if the strike of a fault is at a small angle to the tunnel axis. However, in situations in which the strike of a steeply dipping fault is transverse to the tunnel axis and the tunnel is driven from the footwall side there is a possibility that a wedge-shaped block, will fall from the roof without warning. Major faults are usually associated with a number of minor faults and the dislocation zone, probably containing shattered and unstable material, may occur over many metres.

Tunnelling in soft ground

In soft ground immediate support of the ground is vital and so tunnelling is usually carried out using a shield. In its simplest form this is a cylindrical drum which may have a cutting edge around the circumference. It is pushed forwards as the tunnelling proceeds. Excavation may be performed by either manual or mechanical means, although with rotary cutting head tunnelling

Table 8. *The effect of joint strike and dip orientations in tunnelling*

Strike perpendicular to tunnel acts				Strike parallel to 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					

machines there is a danger of over-excavation in very soft ground below the groundwater table (Robbins 1976). Such problems can be overcome by the use of compressed air working or resorting to a pressurized bentonite slurry shield to support the face. The latter technique is very suitable for driving tunnels in granular ground below the water table but problems arise if boulders or other hard materials are present.

All soft ground moves into the excavation during tunnelling operations and, as well as time dependent movements which occur in cohesive ground, some materials change their characteristics on exposure to air. For instance, some volcanic deposits may disintegrate. The limitation of the effects of ground movement and rock mass alteration requires rapid tunnelling and matching the work methods to the stand-up time of the ground.

Above the water table the stand-up time principally depends on the shearing and tensile strength of the ground whereas below the water table, it also is influenced by the permeability of the material involved (Terzaghi 1950). In non-cohesive ground, including loose sands and some lightly cemented sandstones, an important feature of tunnelling is the resulting distortion of the tunnel lining and settlements of the ground surface. Below the water table such materials may flow into excavations, leading to ground loss and the possible subsidence of an extensive area of ground surface.

Terzaghi (1950) distinguished various types of soft ground behaviour in connection with tunnel excavation work. Firm ground has sufficient shearing and tensile strength to allow a tunnel heading to be advanced without support. Typical materials of this type include stiff clays with low plasticity and loess above the water table. In ravelling ground, blocks fall from the roof and sides of the tunnel some time after the ground has been exposed. This may occur, perhaps within a few minutes of excavation but if the strength of the ground reduces with time it occurs later. Usually strength reduction arises because of progressive failure mechanisms and changes in pore water pressures. Rapid ravelling is common in residual soils and sands having a clay binder situated below the water table. These materials undergo slow ravelling when situated above the water table.

In running ground the removal of support produces a movement which, under natural conditions, will propagate until the angle of rest of the material involved is attained. Runs take place most commonly 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 is usually preceded by ravelling. Flowing ground is capable of viscous liquid flow. It can invade a tunnel from any angle and may lead to rapid and total infilling of an excavation. Flowing conditions may occur in ground below the water table where the effective grain size exceeds 0.005 mm. Such ground above the water table exhibits either ravelling or running behaviour.

The slow creep deformation of ground, or squeezing, there are no fractures propagated and the ground may not appear to have increased in water content. Many soft and medium clays, shales and highly weathered granites, gneisses and schists display squeezing behaviour. Peck (1969b) showed that the squeezing behaviour of clay in tunnel excavation is related to a stability factor (N) which is related to the overburden pressure, the air pressure above atmospheric in the tunnel and the undrained shear strength of the clay. The effects of the value of this stability factor on tunnel stability are listed in Table 9.

Certain types of ground are prone to volume increases, for instance due to the migration of water into the ground near the excavation. Such conditions develop in overconsolidated clays with a plasticity index in excess of about 30% and in certain shales and mudstones, especially those containing montmorillonite. Yuzer (1982) reported that swelling pressures up to 12 MPa were associated the hydration of anhydrite encountered during tunnelling through an evaporitic formation in Turkey. Generally, it is necessary to allow the swelling to cease before constructing the permanent lining.

In well constructed tunnels most deformation of the soils occurs during construction, before a relatively stiff lining is erected. Temporary support systems for soft ground tunnelling commonly take the form of a shield, with or without face support, fluid pressure from air or bentonite slurry, or a combination of these. According to Farmer & Attewell (1975), in near-surface soft ground tunnelling, deformation occurs mainly through unavoidable intrusion of yielding (undrained) clay or running sand into the face of a tunnelling shield or drum digger and into the annular space around the permanent tunnel lining before grouting. Support systems that restrict the relief of residual stresses must be capable of sustaining high loads, whether in hard or soft materials.

Difficult tunnelling conditions may arise if several materials of differing hardness occur in the face at the

Table 9. *Relationship between stability factor (N) and the squeezing behaviour of clay in tunnel excavations*

N	
1–4	Squeezing adds to the loads on tunnel support systems. Progressive increase with time
4–5	Rate of squeeze liable to cause a problem during excavation
5–6	Squeezing sufficiently rapid to close the annular void created by the tailskin of a shield leading to surface subsidence
6–7	Shear failure ahead of the tunnel causes ground movements into the face
>7	Ground over-stressed to the extent that control of shield becomes difficult

same time. Soft ground over rock in the tunnel invert generally means slow, difficult tunnelling progress by traditional methods and pipe-jacking would probably be entirely precluded. Corestones or boulders within a soft matrix, flints in chalk, quartz inclusions in schists and ironstone nodules in shales are all examples of situations in which such problems may arise. Large boulders, especially, can be difficult to handle unless they are first reduced in size by a jackhammer or by blasting.

Water and tunnels

The construction of tunnels and other underground openings in water-bearing ground poses many problems. It is necessary to ensure that adequate pumping capacity is on hand to deal with the steady inflow of water in excavations and also to cope with any sudden and unexpected water inflows which would otherwise result in the flooding of the works. Isolated heavy flows of water commonly occur in association with faults, karst features, abandoned mine workings, or from pockets of high permeability ground, such as saturated gravels. Generally, the amount of water flowing into a tunnel decreases as construction progresses and the water pressures in the area are drawn down. However, if the construction operations, for example blasting, cause increased fracturing of the ground, the flow may increase. The drilling of probe and investigation holes may allow the flow of water from distant sources.

Correct estimation of the water inflow into a projected tunnel is of vital importance, as high inflows usually hinder tunnelling operations. Besides the danger of flooding, the stability of the excavations are liable to be adversely affected. Secondary problems include removal of excessively wet muck and problems with the placement of the lining. Fluctuations in the groundwater pressures due to surface ingress, pumping from wells and other causes may also lead to difficulties with tunnel construction below the water table.

Where flows into underground excavations are excessive, measures to control water ingress, including the general lowering of groundwater levels in the area and treatment of the ground, may be undertaken. Ground treatments such as grouting or freezing are used where very wet and unstable or flowing ground conditions are encountered. Such work is more economically carried out from the surface than from the actual tunnel excavations. Ground freezing, in particular is a very expensive operation which is not normally resorted to except for short lengths or to recover a face that has failed. In addition, excessive groundwater ingress and ground instability can be controlled by carrying out the excavations under

Table 10. *Effects of noxious gases (after Anon 1973)*

Gas	Concentration by volume in air p.p.m.	Effect
Carbon monoxide	100	Threshold Limit Value under which it is believed nearly all workers may be repeatedly exposed day after day without adverse effect (T.L.V.)
	200	Headache after about 7 hours if resting or after 2 hours if working
	400	Headache and discomfort, with possibility of collapse, after 2 hours at rest or 45 minutes exertion
	1 200	Palpitations after 30 minutes at rest or 10 minutes exertion
	2 000	Unconsciousness after 30 minutes at rest or 10 minutes exertion
Carbon dioxide	5 000	T.L.V. Lung ventilation slightly increased
	50 000	Breathing is laboured
	90 000	Depression of breathing commences
Hydrogen sulphide	10	T.L.V.
	100	Irritation to eyes and throat: headache
	200	Maximum concentration tolerable for 1 hour
	1 000	Immediate unconsciousness
Sulphur dioxide	1–5	Can be detected by taste at the lower level and by smell at the upper level
	5	T.L.V. Onset or irritation to the nose and throat
	20	Irritation to the eyes
	400	Immediately dangerous to life

Notes: 1. Some gases have a synergic effect, that is, they augment the effects of others and cause a lowering of the concentration at which the symptoms shown in the above table occur. Further, a gas which is not itself toxic may increase the toxicity of one of the toxic gases, for example, by increasing the rate of respiration; strenuous work will have a similar effect. 2. Of the gases listed carbon monoxide is the only one likely to prove a danger to life, as it is the commonest. The others become intolerably unpleasant at concentrations far below the danger level.

compressed air conditions. Owing to the physiological effects of compressed air working, the method cannot normally be used at depths in excess of about 15 m below groundwater level. The loss of air into a permeable zone in the face can lead to a blowout and serious flooding of the excavations.

Gases and high temperatures in tunnels

Naturally occurring gas can occupy the pore spaces and voids in rock. Where this is held under pressure it may burst into underground workings causing the rock to fail with explosive force. Many gases are dangerous if encountered in tunnelling operations. For example, methane is an inflammable and explosive gas that occurs in many formations including Coal Measures strata. Other common gases are carbon dioxide, carbon monoxide, sulphur dioxide and hydrogen sulphide all of which are asphixiating or toxic. Carbon dioxide is often associated with volcanic deposits and limestones. The effects of these gases are summarized in Table 10.

The temperature of the ground increases with depth, it is inversely proportional to the thermal conductivity of the material involved and also depends on the local geological situation. Geologically stable areas where the mean temperature increase with depth is about 1°C for every 60 to 80 m contrast with volcanic districts where the same increases may occur over about 10 to 15 m depth. The temperature in a tunnel is also affected by the thermal properties and wetness of the rock and the infiltration of meteoric water, as well as the ventilation and humidity of the air in the excavations. Poor working conditions may arise particularly when sources of hot water and unexpectedly high temperatures are encountered. These situations are most likely to occur in areas of recent volcanic and tectonic activity. Conditions can be improved by increased ventilation, by water spraying and air conditioning.

Foundations

Foundation design is primarily concerned with ensuring that movements of a foundation are kept within limits acceptable to the proposed structure without adversely affecting its functional requirements. The design and construction of the foundation structure requires an understanding of the local geological and groundwater conditions, as well as an appreciation of the various types of problems that can occur.

In order to avoid shear failure or substantial deformation of the ground, foundation pressures should have an adequate factor of safety when compared with the ultimate bearing capacity for the foundation. Although limiting the bearing pressure to a value lower than that which would cause failure of the ground in shear there may still be a risk of excessive settlement. The determination of the allowable bearing capacity of different types of foundations is dealt with in many textbooks on foundation design, for example, Terzaghi (1943), Meyerhof (1951, 1963, 1974), Hansen (1961, 1968), De Beer (1965) and Vesic (1973).

Bearing capacity of soils

Particle size and sorting influence the engineering behaviour of cohesionless sediments. Generally, the larger the particles, the higher is the allowable bearing pressure and deposits consisting of a mixture of different sized particles are usually stronger than those which are uniformly graded. However, the mechanical properties of such sediments depend mainly on their relative density. Densely packed cohesionless deposits have low compressibility, although where a foundation rests below the water table greater settlement is likely to be experienced. A fluctuating water table is likely to lead to increased settlement.

The ultimate bearing capacity of foundations on cohesionless deposits depends on the width and depth of the foundation structure as well as strength properties of the materials concerned and the position of the water table in relation to a foundation structure. A high groundwater table will decrease the ultimate bearing capacity by up to 50%.

Silts, especially those deposited under lacustrine conditions, possess low bearing capacity. Furthermore, protracted large magnitude settlements in these materials are a cause of problems. Loess deposits, particularly those which have not undergone significant weathering, often possess a metastable structure. This leads to collapse due to wetting, especially under load, leading to bearing capacity failure or large settlements. Clemence & Finbarr (1981) provided a summary of the methods of recognizing collapsible soils and predicting their performance.

The ultimate bearing capacity of foundations on clay soils depends on shear strength which, in turn, is influenced by the soil's consistency (Table 11). In

Table 11. *Undrained shear strength of clays*

Consistency	Field characteristics	Shear strength (kPa)
Very stiff	Brittle or very tough	Greater than 150
Stiff	Cannot be moulded in the fingers	75–150
Firm	Can be moulded in the fingers by strong pressure	40–75
Soft	Easily moulded in the fingers	20–40
Very soft	Exudes between the fingers when squeezed in the fist	Less than 20

relation to applied stress, saturated clays behave as purely cohesive materials provided that no change of moisture content occurs and the angle of shearing resistance is equal to zero. 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.

The engineering performance of clay deposits is very much affected by moisture content and by the energy with which this moisture is held. In order to minimize the deleterious effects of moisture movements in cohesive soils, foundations should be placed at depths which are unaffected by seasonal fluctuation of moisture content. Clay deposits in particular are susceptible to swelling or shrinkage. Williams & Jennings (1977) found that soil structure has a major influence on the shrinkage process. A review of the various methods which have been used to determine the amount of swelling that an expansive clay is likely to undergo when wetted has been provided by O'Neill & Poormoayed (1980).

Clay soils also undergo slow changes in volume in response to increases and decreases in loading. Consolidation is initially brought about by a reduction in the void ratio as pore air and water is expelled from the soil but further consolidation may occur due to a rearrangement of the soil particles, and mineralogical changes. Normally consolidated clays are more compressible than overconsolidated ones of the same density (Burland 1990). Furthermore, clays which have undergone volume increase due to swelling are liable to suffer significantly increased gross settlement when they are loaded. Bjerrum (1967) explained that the time-dependent vertical swelling occurs due to localized shear stress failures or localized tensile stress failures. The former process is associated with the long-term deformations of soils having well-developed diagenetic bonds. Although when an excavation is made in a clay with weak diagenetic bonds, elastic rebound will cause vertical expansion of the soil, part of the strain energy will be retained due to the restriction on lateral straining. This can also lead to elevated horizontal pressures. These will decrease with time as a result of plastic deformation of the clay.

The shear strength of an undisturbed clay is often greater than that obtained when it is remoulded and tested under the same conditions and at the same moisture content. The ratio of the undisturbed to the remoulded strength at the same moisture content is referred to as the sensitivity of a clay. Clays with high sensitivity values (over 8) have little or no strength after being disturbed. Sensitive clays (4 to 8) generally possess high moisture contents, frequently with liquidity indices well in excess of unity. A sharp increase in moisture content may cause a great increase in sensitivity,

Table 12. *Presumed allowable bearing values under static loading (after Anon 1986)*

Category	Types of rocks and soils	Presumed allowable bearing value (kPa)	Remarks
Rocks	Strong igneous and gneissic rocks in sound condition	10 000	These values are based on the assumption that the foundations are taken down to unweathered rock
	Strong limestones and strong sandstones	4 000	
	Schists and slates	3 000	
	Strong shales, strong mudstones and strong siltstones	2 000	
Non-cohesive soils	Dense gravel, or dense sand and gravel	>600	
	Medium dense gravel, or medium dense sand and gravel	<200–600	Width of foundation not less than 1 m. Groundwater level assumed to be at a depth not less than below the base of the foundation
	Loose gravel, or loose sand and gravel	<200	
	Compact sand	>300	
	Medium dense sand	100–300	
	Loose sand	<100	
		Value depending on degree of looseness	
Cohesive soils	Very stiff boulder clays and hard clays	300 to 600	Susceptible to long-term consolidation settlement
	Stiff clays	150 to 300	
	Firm clays	75 to 150	
	Soft clays and silts	<75	

Note: These values are for preliminary design purposes only, and may need alteration upwards or downwards. No addition has been made for the depth of embedment of the foundation.

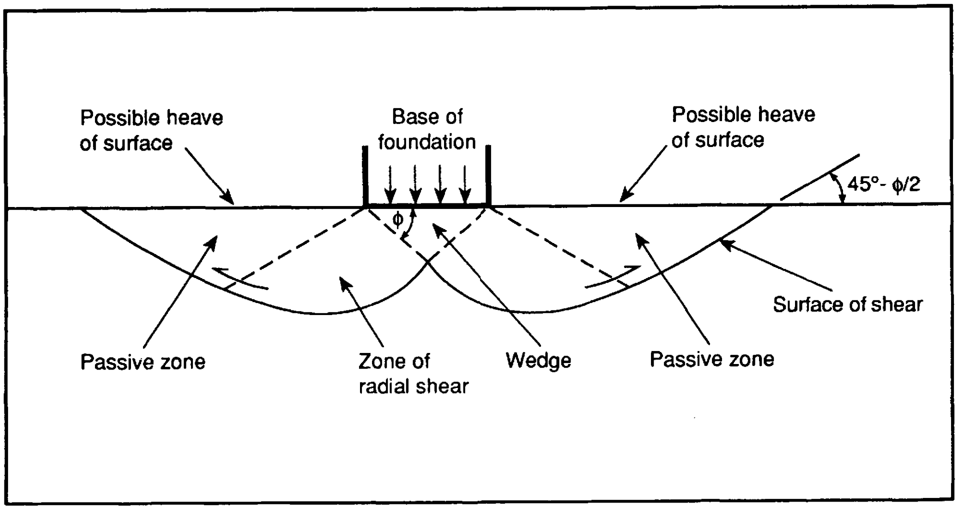

Fig. 6. Typical bearing capacity failure of the soil supporting a foundation.

sometimes with disastrous results. Heavily overconsolidated clays are insensitive (less than 1).

The value of ultimate bearing capacity depends on the type of foundation structure as well as the soil properties. For example, the dimensions, shape and depth at which a footing is placed all influence the bearing capacity. In sandy deposits increasing the width of a strip footing improves the bearing capacity whilst in saturated clays the width has little effect. 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 increased overburden pressure at foundation level and the shear forces that can be mobilized between the sides of the foundation structure and the ground. The presumed bearing values for various types of soil and rock are given in Table 12.

Bearing capacity failure in soils usually occurs as a shear failure of the soil supporting the foundation (Fig. 6). Generally, the mode of failure is related to the geometry of the footing, the loading conditions and the ground conditions, in particular the compressibility of the soil. General shear failure occurs in soils which have low compressibility, such as dense sands and undrained saturated normally consolidated clays. Punching shear occurs in compressible clays in which all the volume change is located beneath the footing. It may also occur if the foundation is situated in a thin incompressible layer which is underlain by a highly compressible one. Where a weak horizon overlies a stronger one, shear will be confined to the weaker material.

In a rock mass containing few defects, the allowable contact pressure at the surface may be taken conservatively as the unconfined compressive strength of the intact rock. Table 13 gives values of allowable contact pressure for jointed rocks in terms of their average RQD values. Using the table, settlements should not exceed 12.5 mm within a depth below foundation level equal to the foundation width. 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, then the design of foundations should be according to the matrix material. Piles or piers can be used to provide support at depth.

Inclined strata complicates into both design and construction of most foundation structures. The presence of faults and shear zones are liable to compromise the suitability of the foundations if they are occupied by weak compressible material.

Table 13. *Allowable contact pressure for jointed rock* (after Peck et al. 1974)*

RQD	Allowable contact pressure (MPa)
100	32.2
90	21.5
75	12.9
50	7.0
25	3.2
0	1.1

* If the value of the allowable contact pressure exceeds the unconfined compressive strength of intact samples, then it should be taken as the unconfined compressive strength.

Types of foundation structure

The design of foundations embodies three essential operations: calculating the loads to be transmitted by the foundation structure to the soil or rock supporting it, determining the engineering performance of the soil and rock, and then designing a suitable foundation.

Footings distribute the load to the ground over an area sufficient to suit the pressures to the soil or rock. Therefore, their size 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 which is at least one and a half times the width of the base. In addition, 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 in moisture content. In the UK, 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 chosen to provide an adequate factor of safety against shear failure in the soil and to ensure that settlements are not excessive. Settlements for any given pressure increases with the width of footing in almost direct proportion on clays and to a lesser degree on sands. By providing a lower contact pressure, a raft permits the construction of a satisfactory foundation in low strength or compressible materials whose strength is too low for the use of footings. Also, rafts provide support in highly variable found conditions where otherwise large differential settlements would occur. In very poor conditions founding the raft at depth below the ground surface means that the increase in vertical stress sustained by the ground is reduced to the difference between the weight of the structure and the weight of the soil removed prior to the construction. 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.

Piles either derive their support in end-bearing or shaft friction or a combination of the two. Friction is likely to be the predominant factor for piles in clays and silts, whilst end-bearing provides the carrying capacity for piles terminating in or on gravel or rock. An important aspect of site investigation for piles rests with the determination of the depth of a suitable bearing stratum and the properties of the ground so far as installation is concerned.

Driving piles leads to vertical and lateral ground displacements of soil which may give rise to heave or compaction which may have detrimental effects upon existing piles and neighbouring structures. The dissipation of energy in the course of pile driving also can have deleterious effects on the geotechnical properties of sensitive clays, chalk and loose saturated sands. In such materials piles are usually formed by cast *in situ* techniques.

Settlement

Excessive total or differential settlement effects the extent to which the structure serves its intended purpose. In cohesive soils settlement may affect neighbouring structures. The great difficulty in assessing the engineering performance of cohesionless soils for foundations arises because of the difficulty of obtaining undisturbed samples. Various empirical methods based on field tests are used (Simons & Menzies 1975). Some settlements are likely in loose sands, particularly below the water table, and especially where the water table fluctuates or the ground is subject to vibrations. However, settlement commonly is relatively rapid in sand and gravel, frequently being substantially complete by the end of the construction period.

Settlement can present a problem in clayey soils and it invariably continues after the construction period. Methods whereby immediate (elastic) settlement can be determined have been provided by Steinbrenner (1934), Terzaghi (1943), Fox (1948) and Janbu *et al.* (1956). Simons & Menzies (1975) described a method of estimating the amount of settlement from the results of laboratory tests on samples of clay soils beneath foundations. The rate at which the long-term settlement occurs depends on the permeability of the soil. The amount and rate of settlement both depend on the expulsion of water (and pore gasses) from the pore space in the soil. Therefore, the settlement is related to the reduction of pore volume. In some soils, for instance organic clays, the solid fraction may also reduce in volume, leading to secondary consolidation. According to Bjerrum (1966), a varying loading condition can lead to secondary consolidation in other materials, including over-consolidated clays. Butler (1975) noted that the standard methods of estimating the degree of settlement in an overconsolidated clay tend to over-estimate the value.

Due to the presence of fissures, many overconsolidated clays have only a tenth of their intact strength. In fact the shear strength developed along closed fissures hardly exceeds the residual value. Fissures tend to open on excavation and through the ingress of water allow the softening of the clay. Owing to concentrations of shear stress, which locally exceed the peak strength, they can also give rise to progressive failure.

Table 14. *Limiting values of distortion and deflexion of structures (after Tomlinson 1986)*

Type of structure	Type of damage	Limiting values			
		Values of relative rotation (angular distortion)			
		Skempton and MacDonald (1956)	Meyerhof (1947)	Polshin and Tokar (1957)	Bjerrum (1963)
Framed buildings and reinforced load bearing walls	Structural damage	1/150	1/250	1/200	1/150
	Cracking in walls and partitions	1/300 (but 1/500 recommended)	1/500	1/500 (0.7/1000 to 1/1000 for end bays)	1/150
		Values of deflexion A/l			
		Meyerhof (1947)	Polshin and Tokar (1957)	Burland and Wroth (1974)	
Unreinforced loadbearing walls	Cracking by sagging	0.4×10^{-3}	L/H = 3: to 0.4×10^{-3}	At L/H = $1{:}0.4 \times 10^{-3}$ At L/H = $5{:}0.8 \times 10^{-3}$	
	Cracking by hogging	—	—	At L/H = $1{:}0.2 \times 10^{-3}$ At L/H = $5{:}0.4 \times 10^{-3}$	

Note: The limiting values for framed buildings are for structural members of average dimensions. Values may be much less for exceptionally large and stiff beams or columns for which the limiting values of angular distortion should be obtained by structural analysis.

Differential and excessive settlement is the principal problem with peaty soil. (Berry *et al.* 1985). Serious shearing stresses are induced even by moderate loads. Primary consolidation, often as 50% of total settlement is likely to take place within the period of construction (Hobbs 1986). Apart from certain weak rocks, settlement is rarely a limiting condition in foundations on most fresh rocks and does not entail special study except in the case of special structures where settlements must be small (Hobbs 1975).

Differential settlement is of greater significance than maximum settlement since the former is likely to lead to distortion and structural cracking. In an examination of differential settlement in buildings, Grant *et al.* (1974) suggested the maximum allowable angular distortions given in Table 14. Generally, some damage will occur in a building which experiences a maximum value of angular distortion greater than 1:300. For most buildings it is relative deflections which occur after completion that cause most damage. Therefore, the ratio between the immediate and total settlement is important. In overconsolidated clay this averages about 0.6 whilst for normally consolidated clay it is usually less than 0.2. Since the total settlement tends to be high, designing foundations for normally consolidated clays is usually more difficult than for overconsolidated clays.

Settlement can be reduced by loading the site prior to construction or by ground treatment methods. To reduce the time for the total settlement to be completed band drains or sandwicks may be installed in the ground to be loaded.

Surface subsidence

Surface subsidence, which occurs as a consequence of the extraction of mineral deposits or the abstraction of water, oil, or natural gas from the ground, may cause foundation problems. Besides subsidence due to the removal of support, mining often entails the lowering of groundwater levels which may lead to consolidation of the ground.

Collapse of the ground may take place many years after mining activity has ceased. Bell-pitting, an early from of shallow mining producing shaft-like excavations extending to depths of about 12 m, leaves an area an area of highly disturbed ground which requires treatment or complete excavation. Otherwise, structures need to be piled to depths below the working. Where the pillar and stall method of excavation has been used, slow deterioration and failure of pillars may take place, leading to pillar collapse. The yielding of a large number of pillars, or of the associated roof or floor measures, can bring about a shallow, broad, trough-like subsidence at the ground surface. Variable closure of old pillar workings at shallow depth can cause difficult foundation problems. Void migration due to the failure of the roof rocks spanning mine galleries between pillars can lead to the sudden loss of support to overlying foundation structures or the appearance of a crown hole at the surface.

The mining of coal by longwall methods leads to total collapse of the workings and transmission of subsidence and, more significantly ground strains, to the surface (Bell 1988a). As longwall mining proceeds the ground is subject to tilting accompanied by tension and then compression. Although once the subsidence front has passed by, the ground attains its previous slope and the ground strain returns to zero, permanent ground strains affect the ground above the edges of the extracted panel.

Subsidence damage to structures on conventional foundations is liable to occur when they are subjected to effective strains of 0.5 mm to 1 mm/m. Bhattacharya and Singh (1985) provided a classification of subsidence induced damage. In many instances subsidence effects also have been affected by the geological structure, notably the presence of faults and the character of the rocks and soils above the workings (Bell & Fox 1991). A review of measures which can be taken to mitigate the effects of subsidence due to coal workings has been provided by Anon (1975) and Anon (1977). Methods of dealing with old shafts have been outlined by Anon (1982).

Deposits that readily go into solution, in particular salt, can be extracted by solution mining (Bell 1992). For example, wild brine pumping formerly caused subsidence associated with tension cracks, and small fault scars have formed at the surface on the convex flanks of subsidence hollows in Cheshire in the UK. Sulphur, mined in Texas and Louisiana, USA, by the Frasch process, has also caused similar problems.

Surface subsidence also occurs in areas where there is intensive abstraction of oil or groundwater, as well as natural gas, from the ground (Bell 1988b). Here, subsidence is attributed to the consolidation of the fluid-bearing formations which results from the increase in vertical effective stress. In most cases, and particularly where clayey deposits occur, the subsidence occurs after the abstraction. In the case of groundwater, a reduction in the rate of abstraction can lead to a rise in the water table. This, in turn, can lead to a rise in the ground surface; for example, in the Venice area of Italy, a rise in the water table has brought about some 20 mm of rebound since 1970. Although controlled withdrawal of groundwater can permit the re-establishment of the natural hydraulic balance, recovery uplift is never complete. Rising water tables can cause reductions in pile capacity, the possibility of structures settling, damage to basement floors, and disruption of utility conduits as well as basement flooding.

Rapid subsidence can take place due to the collapse of cavities within limestone which has been subjected to prolonged solution, this occurring when the roof rocks are no longer thick enough to support the imposed loading (Beck 1984). Many sinkholes develop as a result of declines in groundwater level due to excessive abstraction. Most collapses forming sinkholes result from roof failures of cavities formed in unconsolidated deposits overlying limestones. Dewatering associated with mining in the gold-bearing reefs of the Far West Rand, South Africa, which underlie dolostone and unconsolidated deposits, has led to the formation of some spectacular sinkholes, some of which led to the loss of lives, and produced differential subsidence over a large area (Bezuidenhout & Enslin 1970).

Fills and waste materials

Construction on colliery discard poses special problems. Mine waste material is essentially granular, most falling within the sand range, but significant proportions of gravel and cobble sizes also may be present. Burnt (red) shale is stronger but, on the other hand, with an increasing content of fine coal, the angle of shearing resistance is reduced. The sulphate content of weathered, unburnt colliery waste, due to the breakdown of pyrite, is usually high enough to warrant special precautions in the design of concrete structures which may be in contact with the discard or water issuing from it.

Spontaneous combustion of carbonaceous material, frequently aggravated by the oxidation of pyrite, is the most common cause of burning spoil. The problem of combustion has sometimes to be faced when reclaiming old tips. Anon (1973) recommended digging out, trenching, blanketing, injection with non-combustible material and water, and water spraying as methods by which spontaneous combustion in spoil material can be controlled. Moreover, spontaneous combustion may give rise to subsurface cavities in spoil heaps. Burnt ashes may also cover zones which are red hot to appreciable depths. When steam comes in contact with red hot carbonaceous material watergas is formed and when the latter is mixed with air it becomes potentially explosive. Explosions may also occur when burning spoil heaps are being reworked and a cloud of coal dust is formed near a heat surface. If the mixture of coal dust and air is ignited it may explode violently.

A wide variety of materials are used for fills including domestic refuse, ashes, slag, clinker, building waste, chemical waste, quarry waste and all types of soils. The extent to which an existing fill will be suitable as a foundation material depends largely on its composition and uniformity. Of particular importance is the time required for a fill to reach a sufficient degree of natural consolidation to make it suitable for sustaining the required loading. This depends on the nature and thickness of the fill, the method of placing, and the nature of the underlying ground, especially the groundwater conditions. The best materials in this respect are obviously well graded, hard and granular. Furthermore, properly compacted fills on a sound foundation can be as good as, or better than, virgin soil. Fills containing a large proportion of fine material, by contrast, may take a relatively long time to settle. Similarly, old fills and those placed over low-lying areas of compressible or weak strata should be considered unsuitable unless tests demonstrate otherwise or the proposed structure can be designed for low bearing capacity and large and irregular settlements. Mixed fills which contain materials liable to decay, which may leave voids or involve a risk of spontaneous combustion, afford very variable support and, in general, such sites should be avoided. Some materials, such as ashes and industrial wastes, may contain sulphate and other products which are potentially injurious as far as concrete and other construction materials are concerned.

Sanitary land fills, in particular, suffer from continuing organic decomposition and physico-chemical breakdown with the generation of carbon dioxide, methane and, to a lesser extent, hydrogen sulphide (Oweis & Khera 1990). The gases mentioned are asphyxiates or toxic and methane forms a highly explosive mixture with air (Williams & Aitkenhead 1991). Bell & Wilson (1988) described various chemical and biochemical hazards.

Many urban renewal schemes require the construction of foundations in areas covered by fill, contaminated or previously developed land. The ground conditions can be very variable with zones or compressible fill and other materials. With demolition sites the most economical method of constructing foundations is usually either to cut a trench through the fill and backfill it with lean concrete or to excavate the fill. Ground consolidation by vibration, dynamic compaction, preloading or grouting techniques may also be used.

Grouting refers to the process of injecting setting fluids into fissures, pores and cavities in the ground (Cambefort 1987). It may either be pre-planned, or an emergency expedient. The process is widely used in foundation engineering in order to reduce seepage of water or to increase the mechanical performance of the soils or rocks concerned. Generally, cement grouts are limited to soils with pore or fissure dimensions greater than 0.2 mm (Table 15). Chemical grouts are used in finer materials (Littlejohn 1985*a*, *b*, *c*). Cavities in rocks may have to be filled with bulk grouts (usually mixtures of cement, pulverized fly-ash, sand and gravel), or foam grouts.

Vibration of an appropriate form can eliminate intergranular friction of cohesionless soils so those initially loosely packed can be converted into a dense layer (Brown 1977). The best results have been obtained in fairly coarse sands which contain little or no silt or clay. Vibroreplacement, in which columns of granular material are placed by vibratory methods are commonly used in soft, normally consolidated compressible clays, saturated silts, and alluvial and estuarine soils (Hughes & Withers 1974).

Table 15. *Types of grout (after Anon 1986)*

Ground	Typical grouts used	Examples
Alluvials		
Open gravels Gravels Coarse sands Medium sands	Suspension	Cement suspensions with particle size of about 0.55 mm Cement clay, clay treated with reagents Separated clay and reagents, *bentonitic* clays with sodium silicate and deflocculants (clay gels) Two-shot sodium silicate based systems for conferring strength Bituminous emulsions with fillers and emulsion breaker
Coarse sands Medium sands	Colloidal solutions	Single-shot silicate based systems for strength (silicate–organic ester) Single-shot lignin based grouts for moderate strength and impermeability Silicate–metal salt single-shot, e.g. sodium silicate–sodium aluminate; sodium silicate–sodium bicarbonate Water soluble precondensates, e.g. urea–formaldehyde Oil based clastomers (high viscosity)
Fine sands Silts	Solutions	Water soluble polysaccharides with metal salt to give insoluble precipitate Water soluble acrylamide, water soluble phenoplasts
Fissured rocks		
Open jointed Medium jointed	Suspensions	Cement–sand, cement, cement clay
Medium jointed Fine jointed	Solutions	Oil based elastomers, non-water soluble polyesters, epoxides, and range of water soluble polymer systems given above Hair cracks in concrete would be treated with a high strength low viscosity polyester or epoxy resin

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 (Menard & Broise 1975; Gambin 1987). This is achieved by dropping a large weight, typically 10 to 20 tonnes, from crawler cranes, from heights of 15 to 40 m, at regular intervals across the surface. Repeated passes are made over a site, although several tampings may be made at each imprint during a pass. Each imprint is backfilled with granular material.

Highways

Since the construction of highways includes the excavation of soils and rocks, the provision of stable foundations for the road itself, bridges and other structures and the building of embankments, many aspects have been covered in the previous sections. However, there are some the particular features of highways that require attention.

Since in common with tunnels and other elements of the infrastructure, highways are linear structures, they often need to traverse a wide variety of ground conditions along their length. Due to the long, narrow nature of the site, the ground is less easy to investigate (Dumbleton & West 1976) and access for equipment and materials during construction can be constricted. If even slightly adverse ground conditions follow the route of the construction they are liable to have a profound effect upon the success of the project. For example, siting a highway on a soliflucted valley side rather than on the more stable valley floor is liable to cause considerable avoidable problems with ground stability (Weeks 1969).

It is usual to use the steepest side slopes possible when constructing cuttings and embankments. This minimizes the land required for the structure and the amount of material to be moved. Table 16 shows common slope angles for materials of different types.

As indicated in the section on excavations above, the stability of a cut slope depends on many factors and the use of standard slope angles could be very misleading. In hard rocks, rock mass discontinuities, including bedding surfaces, joints and cleavage, may exert an over-riding influence on the stability of a cutting. The water conditions may also exert a strong control over the stability of a slope. Hoek & Bray (1981) described methods of assessing the stability of slopes in jointed rock masses (see also Matheson 1989). In practice, attention needs to be given to the assessment of the long-term stability of the rock mass since, although stable at the time of construction, a slope may later cause problems.

Table 16. *Suggested slope angles for highway cuttings and embankments (after Ashworth 1972)*

Material	Slope type	
	Cutting	Embankment
Igneous and competent sedimentary rocks	1 in 0.25	1 in 1 to 1.5
Slates, marls and shades	1 in 0.5 to 0.75	1 in 1.5
Gravel	1 in 1	1 in 1.5
Sand	1 in 1.5 to 2	1 in 1.5 to 2
Clay	1 in 2	1 in 2 to 4

At the time of construction attention needs to be given to the effect discontinuities will have on the ease of excavation. It is likely that either under- or over-excavation will take place if a strongly formed set of discontinuities lies at an angle of less than about 15° to the required slope profile. In sedimentary rocks the most stable configuration is where the strike is perpendicular to the face, since then there is a low tendency for slip of rock slabs along the bedding planes. Where the strike of rocks parallels the slope face there will be a tendency for slabs of rock to slide into the cutting. In rock sequences containing mudrocks, slope angles as low as 10° may need to be used. Where there are existing shear surfaces, as for instance in many soliflucted materials, or where adverse groundwater conditions are present the slopes may need to be at a shallower angle.

Guidance as to the method of excavation appropriate for rocks having different strengths and discontinuity spacings is given above. Difficulties frequently occur in heterogeneous materials and those that are marginal between soils and rocks. Slight variations in the degree of cementation, spacing of fractures or the weathering grade can have a profound effect on the rate at which material may be excavated and may also affect its suitability for use as a construction material. Bands of harder rock within a sequence of rippable materials are a frequent cause of difficulty.

Where blasting is required for excavation, care needs to be taken that the rock mass which will form the side slopes in the cutting is not damaged in the process. Wholesale bulk blasting fractures the rock mass and opens discontinuities such that side slopes become unstable soon after construction. The pre-slit method of blasting may be used to reduce the damage to the rock mass (Matheson 1989).

A typical highway project will include an assessment of the suitability of the material excavated in the course of forming cuttings for use as embankment fill and other purposes. For instance, in providing specifications for materials for highway construction in the UK, Anon (1991) distinguished between general granular, cohesive and chalk fills, landscaping fills and selected granular fills. Depending on the type of material and its application within the project, ranges of allowable values for various physical and chemical attributes are specified. Various materials, including peat, wood and frozen, organic and very clayey soils are specifically excluded. Attention is drawn particularly to the possibility of the deterioration of chalk for fill due to poor handling during excavation and subsequent use. Where the materials excavated are not suitable for use during construction, then considerable extra expense may be entailed in disposing of waste and importing fill.

The ground beneath embankments and roads must have sufficient bearing capacity to prevent foundation failure and also be capable of preventing excess settlements due to the imposed load. Very weak and compressible ground may need to be entirely removed before construction takes place. 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 be carried out prior to embankment and road building.

Geological features such as faults, crush zones and solution cavities, as well as artifacts such as mines, shafts, drains and areas of fill can cause considerable difficulties with construction. This is particularly so if a linear feature is orientated parallel to the line of highways. Care needs to be taken that such adverse features are avoided or crossed by as short a route as possible.

Although as indicated in Table 16, sands and other materials are usually stable in slopes of up to about 1 in 1.5 (30 to 40°), this applies only if the slope is well drained. Care should be taken that instability is not instigated by oversteepening during construction. Also, a lack of permanent drainage structures can make slopes vulnerable to the adverse effects of storm events during the construction period. Some uncemented sands and silts, in particular, are liable to liquefy if they become saturated. Pulverized fuel ash and loess are two materials that are particularly vulnerable to erosion by uncontrolled surface run-off during heavy rainfall and many fine grained soils and chalk are susceptible to damage by frost action.

Oversteepening or the presence of high pore water pressures during construction can also have a permanent detrimental effect on the performance of slopes formed in clays and mudrocks or sequences that include these materials. The establishment of any shear surfaces and the opening of tension cracks at this stage would diminish the long-term stability of the slope. Clays and mudrocks are also materials that are susceptible to deterioration due to changes in water content during construction. Alternate wetting and drying or freezing and thawing are liable to lead to complete disintegration of the material. Vulnerable materials may need to be covered with blinding concrete to prevent such deterioration. The presence of pyrite within mudrocks may also need to be considered since on exposure to the atmosphere, oxidation, sometimes aided by the activities of bacteria, can result in the generation of acidic ground waters and adverse chemical reactions with construction or geological materials. Hawkins & Wilson (1990) drew attention to the possibility of the oxidation of pyrite giving rise to high sulphate values in both fills and *in situ* materials.

The heave which occurs due to the removal of load may cause significant problems in some cases. Clay deposits, in particular, undergo rapid elastic strain recovery followed by longer-term swelling. Heave increases the amount of settlement that occurs once the material is reloaded and may lead to the distortion of a road surface or other structures such as drains.

Concluding comments

Problems liable to arise during construction should be given adequate attention during the site investigation stage. Although considerable effort is devoted to acquiring the information required for the design of structures, due to a lack of appreciation for the method of construction to be used there can be a lack of attention to potential problems during construction. This is particularly relevant to infrastructure projects in which much effort is taken to investigate the sites of individual structures such as bridges or tunnel portals, but less attention is devoted to the intervening areas of road or tunnel construction.

Leaving uncertainty about the engineering geological conditions can lead to a over-optimistic view of the ground conditions being taken. Since contractors who offer to carry out the work at the lowest price are more likely to receive the contract, contractors are encouraged in this direction. In areas of complex geology, the site investigation may not yield sufficient information for an accurate model of the ground conditions to be derived. However, the implications of uncertainty on the methods of construction need to be fully considered. If the risk of delay and extra costs during construction are very high, a further stage of investigation, for example a pilot construction, or a complete change in the design may be justified.

Acknowledgements. This paper is published with the permission of the Director of the British Geological Survey (NERC).

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