6 Analysis, design and construction

6.1 Introduction

In Chapter 2, a brief introduction was given to civil engineering practice and types of structure. This chapter provides more detail so that the engineering geologist can better understand the requirements of projects, in terms of site investigation, design and construction issues.

6.2 Loads

Most civil engineering projects involve either loading the ground, say from the weight of a new building, or unloading because of excavation of a slope or in a tunnel. Load changes can be permanent or temporary, static (due to weight) or dynamic (due to blasting, for example). A further important consideration for most geotechnical problems is the self weight of the ground and other *in situ* stresses.

6.2.1 Natural stress conditions

At any point in the Earth's crust, the stresses can be resolved into three orthogonal directions. These are termed the maximum, intermediate and minimum principal stresses and depicted σ_1, σ_2 and σ_3, respectively. By definition, the planes to which the principal stresses are normal are called principal planes and the shear stresses on these planes are zero. An important point regarding rock engineering is that all unsupported excavation surfaces are principal stress planes because there are no shear stresses acting on them (Hudson, 1989). One of the principal stresses will always be perpendicular to the Earth's surface (Anderson, 1951) and is generally vertical.

For projects close to the Earth's surface, such as cut slopes or foundations, natural stresses include self weight, weight of included water and buoyancy effects below the water table, which reduces the total stress to an effective stress (weight of soil minus water pressure), as illustrated in

Box 6-1. As the rock or soil is compressed under self weight, it tries to expand laterally and a horizontal stress is exerted. This is termed the Poisson effect. Typically, in a soil profile at shallow depths (tens of metres), the *in situ* horizontal stress (σ_h) due to self weight will be between about 0.3 (in loose sand) and 0.6 times (in dense sand) the vertical gravitational stress. The value 0.3 to 0.6 is called the coefficient of earth pressure at rest. In normally consolidated clay, the value is about the same as for dense sand: 0.6. For most rocks, the Poisson's ratio is slightly less than 0.3. Most continental rocks weigh about 27 kN/m^3, so at a depth of 500m the total vertical stress can be anticipated to be about 13.5 MPa, and horizontal stresses (σ_h) about 4 MPa.

Box B6-1 Example stress calculations

Generally, stresses are estimated by calculating the total weight of a vertical column of soil based on unit weight measurements. Effective stress is estimated by subtracting measured or estimated water pressure from the total stress due to the bulk weight of the soil or rock (including contained water).

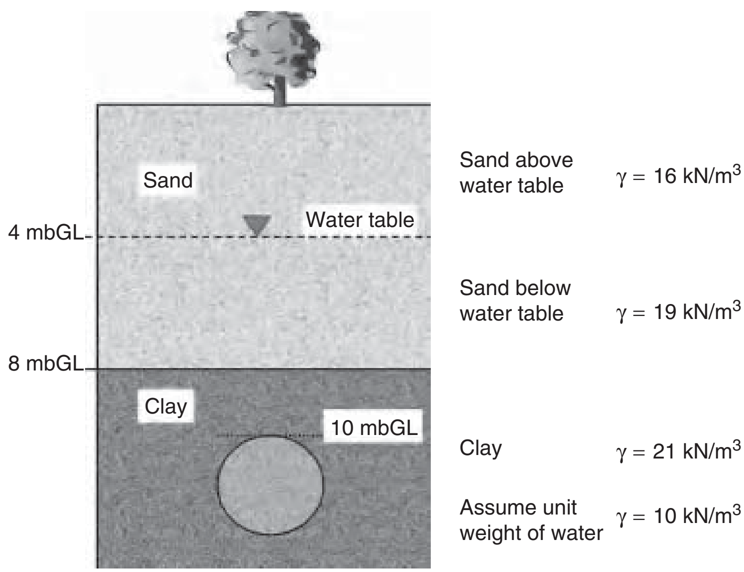


Figure B6-1.1 Soil profile with tunnel to be constructed with crown at 10mbGL

In Figure B6-1.1, a ground profile is shown with sand overlying clay. The water table (upper surface of saturated ground) is 4m below ground level (mbGL).

The unit weight (γ) of the damp sand above the water table is 16 kN/m^3; the unit weight below the water table, sand plus pores full of water (γ_{sat}), is 19 kN/m^3. The underlying saturated clay has unit weight γ_{sat} = 21 kN/m^3. The unit weight of fresh water, γ_w, is about 9.81 kN/m^3 (10 is generally a near-enough approximation given other assumptions).

We wish to estimate the vertical stress at the crown of a tunnel to be constructed at a depth of 10 mbGL. As shown in Figure B6-1.2.

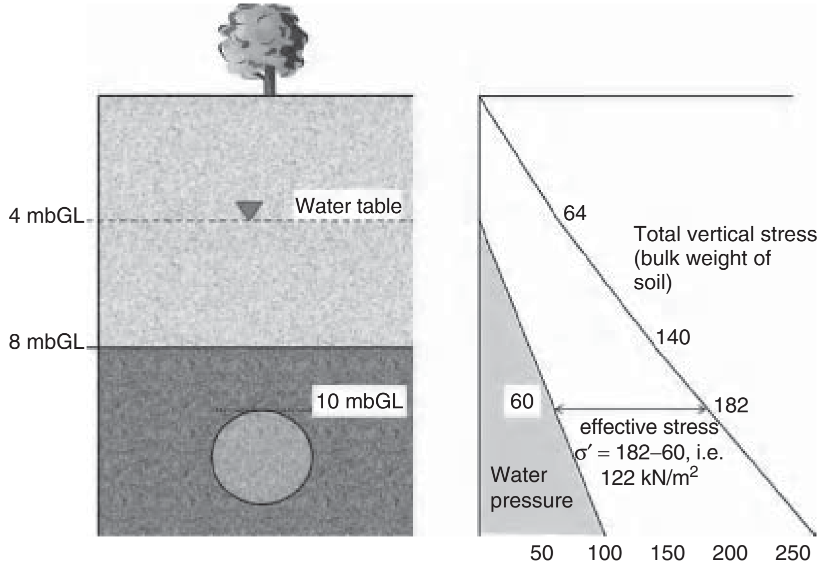


Figure B6-1.2 Total and effective stresses(vertical)

At depth	*Total vertical stress* $(\sigma_v)kN/m^2$	*Water pressure (u)* kN/m^2	*Effective vertical stress* $(\sigma')kN/m^2$
4m	$4 \times 16 = 64$	0	64
8m	$64 + (4 \times 19) = 140$	$4 \times 10 = 40$	$140 - 40 = 100$
10m, at tunnel crown	$140 + (2 \times 21) = 182$	$6 \times 10 = 60$	$182 - 60 = 122$

Therefore, before tunnel construction, the estimated vertical effective stress at the tunnel crown is 122 kN/m^2. During construction, due to seepage into the tunnel the water table would be lowered or this might be done deliberately to excavate 'in the dry' to avoid flowing or ravelling of the soil into the tunnel. If the water pressure dropped, so the effective stress would increase. If the water table was lowered so that water pressure was zero at tunnel crown level, then the effective stress would equal the total stress (= 182 kN/m^2).[1]

At some locations, however, tectonic or topographic stresses can be dominant even very close to the Earth's surface, with horizontal stresses sometimes locked in from a previous geological event and far in excess of that due to gravity and the Poisson

[1] Note: the actual stress conditions near a tunnel would be more complex than this calculation. The tunnel would distort the stress field – refer to Muir Wood (2000) or Hoek *et al*. (1995).

effect. As illustrated in Box 6-2, in overconsolidated clays such as London Clay, where the rock has been buried to considerable depth before uplift, erosion and unloading, then the earth pressure at rest can be up to three times the vertical stress. In tectonically active regions, stresses can be higher or lower than lithostatic. Horizontal: vertical stress ratios as high as 15 have been measured in areas where tectonic or thermal stress has been locked in as the overburden has been eroded (Hoek & Brown, 1980). These stresses can adversely affect engineering projects, resulting in deformation in tunnels, rock bursts and propagation of fractures (e.g. Karrow & White, 2002). In mountainous terrain, principal stress trajectories will follow the topography so that the maximum principal stress runs parallel to steep natural slopes, and this leads to spalling off of the rock parallel to the natural slope (Chapter 3) and valley bulging at the toe of the slope.

Box B6-2 Variations from lithostatic stress conditions

Whereas in many areas of the Earth's crust, stress conditions can be estimated reasonably well by calculating the weight of the soil/rock overburden to give vertical stress and taking account of Poisson's effect for horizontal stress, considerable variation is found (Hoek & Brown, 1980). In particular, horizontal stresses can be higher or lower than anticipated.

Example 1 Overconsolidated clay

Soils and weak rocks that have gone through a cycle of burial, partial lithification and then uplift and erosion are termed overconsolidated. They typically have lower void ratios (percentage of pores) and are stiffer than would be expected for normally consolidated soils at similar depths of occurrence. They are also sometimes partially cemented, as described in Chapters 1, 3 and 5. Under compression, they demonstrate high moduli up until the original maximum burial stress, at which point they revert to the normal consolidation stress curve, as described in soil mechanics textbooks (e.g. Craig, 1992). Because the stress level has been much higher in geological history, the horizontal stress may have become locked-in as a residual stress and may be much higher than the vertical principal stress, as illustrated in Figure B6-2.1. Craig quotes earth pressure at rest K_0 values up to 2.8 for heavily overconsolidated London Clay. Further discussion of earth pressures and how they relate to geological history is given by Schmidt (1966).

Example 2 Active and ancient tectonic regions

Deviations from lithostatic stress conditions can be anticipated at destructive plate margins, as along the western margins of North and South America where high horizontal stresses are to be expected. Conversely, in extensional tectonic zones the horizontal stresses can be anticipated to be tensile. Variations can also be expected in ancient mountain chains or areas of igneous intrusion where relict horizontal stresses can be very high, resulting in rock bursts and large deformation of structures (e.g. Holzhausen, 1989).

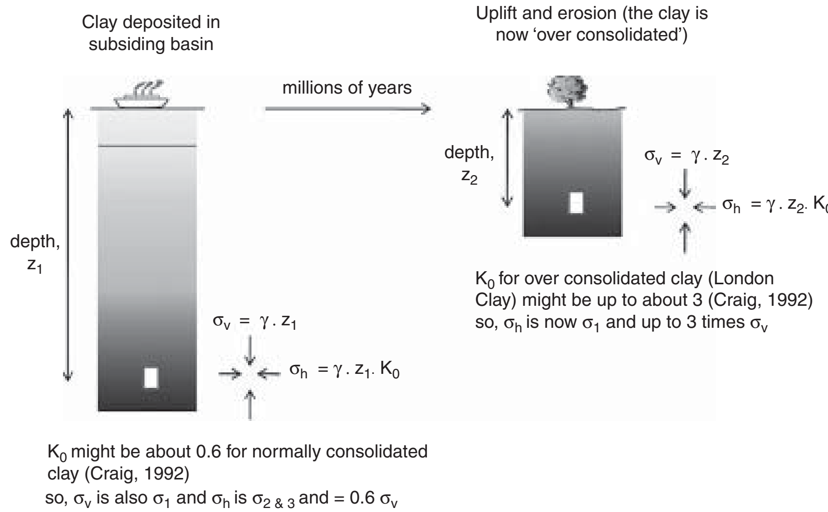


Figure B6-2.1 Stress conditions in overconsolidated soil. Uplift and erosion will result in a reduction in the vertical stress on the soil element but some residual horizontal stress may be retained from its burial history.

Example 3 Topographic stresses

Stress conditions may be strongly affected by local topography exacerbated by geological conditions. At an extreme scale, large-scale mountain structures are ascribed to gravity gliding (e.g. Graham, 1981) and certainly large landslides have ample evidence of compression and tensile zones. Other key examples of the effect of localised topographic stress are sheeting joints (Hencher *et al.*, 2011) and valley bulging (Parks, 1991).

Stress conditions have been measured across the world from instruments, by interpretations of breakouts in deep drillholes for oil and gas exploration, or analysis of earthquakes, and many such data are compiled centrally and are freely available at http://www.world-stress-map. *In situ* stresses are sometimes investigated specifically for projects (Chapter 4) but this is expensive and can be inconclusive because of the small scale and localised nature of tests.

Where stress assumptions prove wrong, the consequences can be severe, as at Pergau Dam, Malaysia, where it had been anticipated that stresses would be lithostatic (i.e. caused by self weight). During construction, open joints and voids were encountered in tunnels together with high inflow of water (Murray & Gray, 1997). It was established that horizontal stresses were much lower than had been expected and this necessitated a complete redesign of shafts and high-pressure tunnels and their linings, at considerable cost. Low horizontal stresses can occur in the proximity of valley sides. Further examples are given later in this chapter.

6.2.2 Loadings from a building

A structure will change the stresses in the ground and, in turn, be acted upon by stresses from the ground due to gravity and tectonic forces, wind, snow, earthquakes and perhaps from anthropogenic sources, including blasting and traffic. The loading condition for a high-rise building constructed on piles is illustrated in Figure 6.1. It is the task of the geotechnical team, given the loading conditions from other members of the design team, to ensure that there is an adequate Factor of Safety for the foundations against failure and that settlement is within the tolerance of the structure. The traditional permissible stress approach, involving a lumped Factor of Safety to cover all uncertainties, has been replaced in Europe and some other countries and design codes by a limit state approach, which encourages more rigorous consideration of different modes of failure and uncertainties in each parameter and in the calculation processes itself (Table 2.2).

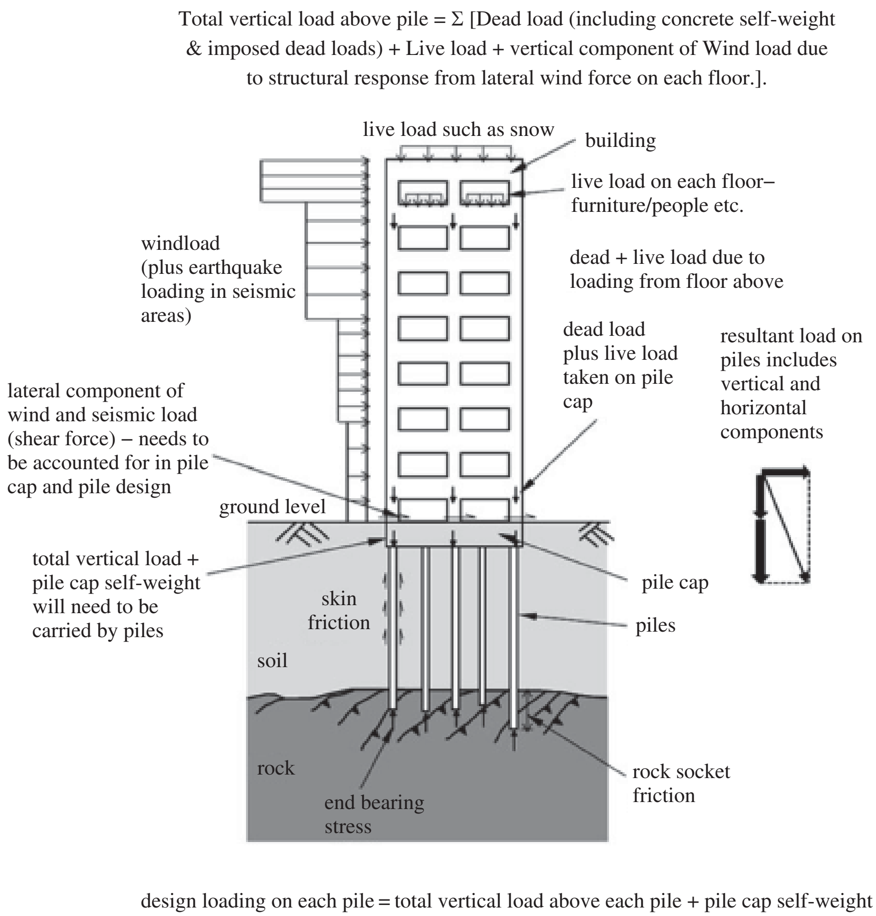


Figure 6.1 Typical loading conditions for a high-rise building to be founded on piles.

6.3 Temporary and permanent works

The engineer's design generally concerns the permanent works – the long-term stability and performance of the finished project. Performance is measured by criteria specific to a project, such as settlement, leakage, durability and long-term maintenance requirements. During construction, there will usually be other design considerations including stability of temporary excavations, disturbance to the groundwater conditions and water inflow to the works. Temporary work design is generally the responsibility of the contractor and his design engineers, perhaps checked by an independent checking engineer. The design of deep temporary excavations can be just as demanding as for permanent works, as illustrated in Figure 6.2. Catastrophic failure of such works is unfortunately common – in recent years affecting such high-profile projects as the International Finance Centre in Seoul, Korea, and the Nicoll Highway subway works in Singapore (Chapter 7). In both cases, the strutted excavations collapsed. Guidance on the design of such structures is given in Puller (2003) and GCO (1990).

In tunnels, during construction there may be a need to stabilise the walls and possibly the working face using rapidly applied techniques, including shotcrete with mesh or steel fibres, steel arches or lattice girders and rock bolts (Hoek *et al.*, 1995). Such measures are generally specified and installed by the contractor, typically agreed with a supervising engineer who may well be an engineering geologist. The engineering geologist will probably be involved in identifying the rock

Figure 6.2 Temporary works for an underground station construction in Singapore. Piles to the left were excavated by a large-diameter drilling rig and then concreted. As excavation has proceeded, the piles have been anchored back into the ground and strutted using systems of waling beams (horizontal, along the face of the piles) and struts, supported where necessary by additional king posts.

mass conditions and identifying any geological structures that might need specific attention, as discussed later. The decisions taken will often have cost as well as safety implications. Usually, measures installed to allow safe working will be ignored when designing and constructing permanent liner support, but in some tunnels there is no permanent lining so the temporary measures also become permanent works. In the latter case, the materials and workmanship will be specified accordingly and as appropriate to the design life of the project. Close supervision will be required on site to ensure that the specified requirements are met and the quality of the works is not compromised.

6.4 Foundations

Foundations are the interface between a building and the ground and transfer loads from the building to the underlying soil and rock. Detailed and practical guidance on foundation design and construction issues is given by Tomlinson (2001). Wyllie (1999) deals specifically with foundations on rock. If ground conditions are suitable, then shallow foundations are used because of cost considerations. These include strip footings beneath the walls of a house (Figure 6.3), pads beneath columns for a steel or concrete-framed structure, or a raft supporting several loading columns and walls.

6.4.1 Shallow foundations

For traditional design involving a single Factor of Safety, which is probably the easiest to understand and still employed as the

Figure 6.3 Concrete strip foundations on weathered limestone for a house, Portugal.

standard approach to design in many parts of the world, the following definitions are used:

Bearing pressure	The net loading pressure: load from structure, divided by the area of the foundation, minus the weight of material removed from the excavation.
Ultimate bearing capacity	The loading pressure at which the ground fails. This is the same as the ultimate limit state in the limit state approach (Eurocode 7).
Allowable bearing pressure	The maximum loading pressure that meets two criteria: 1. An adequate Factor of Safety against failure. 2. Settlement within tolerance of the structure (specific to the particular structure).
Presumed bearing pressure	A net loading pressure considered appropriate for a given ground condition, based usually on local experience and incorporated in building regulations or codes of practice such as BS 8004 (UK) (BSI, 1986) and CP4 (Singapore Standard, 2003).

Typical values are presented in Table 6.1 and can be used for preliminary design purposes. They allow the practicability of foundation options to be assessed and to select appropriate ground investigation, testing and design methods. Presumed values are only appropriate if the site is approximately level (not, for example, at the top of a steep slope) and where the geology is relatively uniform and isotropic with no lenses or layers of significantly weaker or compressible material within the zone of ground that will be stressed. Such tables are generally very conservative and economies can be made by conducting more detailed characterisation with testing and analysis, although sometimes regulating bodies (building authorities) may be loathe to allow higher values to be used without considerable justification.

In Europe, since 2010, Eurocodes have replaced national standards and should be used for design (BSI, 2004). The ultimate limit state (ULS) is essentially the same as ultimate bearing capacity but with possible failure modes spelt out, including sliding resistance and structural capacity, heave, piping, and so on, which were implicit in the BS 8004 approach as factors that a responsible geotechnical engineer should consider. The serviceability limit state (SLS) of Eurocode 7 is defined as: ‘states that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met’, and this equates effectively to the idea of allowable bearing pressure, as far as settlement is concerned, but includes other considerations such as vibration annoyance to neighbours, and so on – again, factors that would usually be considered automatically by experienced and responsible geotechnical engineers when adopting a traditional approach to design.

From Table 6.1 it can be seen that, for rock, the two governing parameters are generally taken to be uniaxial compressive strength (UCS = σ_c) and degree of fracturing. This is expressed in charts presented

Table 6.1 Examples of presumed bearing pressures. These values, which can be used for option assessment, are a selection of more extensive recommendations given in Tomlinson (2001) and BS 8004 (BSI, 1986).

	Examples of rock type (indicative only)		**Presumed bearing value (MPa)**	
ROCK	Bearing on surface of rock		Strip footings < 3 m wide. Length not more than ten times width	
	Strong. Discontinuity spacing more than 200 mm Strong. Discontinuity spacing 60–200 mm Moderately strong. Discontinuity spacing 60–200 mm		10–12.5 5–10 1–5	
	Notes: Figures given are for igneous rocks, well-cemented sandstone, mudstone and schist/slate with flat-lying cleavage/foliation. For other rock types see references quoted. Strength definitions are from BS 5930:1999. Strong rock (σ_c = 50–100 MPa) requires more than one hammer blow to break. Moderately strong rock (σ_c = 12.5–50 MPa) – intact core cannot be broken by hand.			
	Examples of soil type (indicative only)		**Presumed bearing value (MPa)**	
SOIL	Sand and gravel: foundations at least 0.75 m below ground level	SPT N-value	Foundation width	
			<1 m	<2 m
	Very dense Dense Medium dense Loose	> 50 30–50 10–30 5–10	0.8 0.5–0.8 0.15–0.5 0.05–0.15	0.6 0.4–0.6 0.1–0.4 0.05–0.1
	Clay: foundations at least 1 m below ground level	Undrained shear strength (MPa)	Foundation width	
			<1 m	<2 m
	Hard Very stiff Stiff Firm Soft	> 0.30 0.15–0.30 0.075–0.15 0.04–0.075 0.02–0.04	0.8 0.4–0.8 0.2–0.4 0.1–0.2 0.05–0.1	0.6 0.3–0.5 0.15–0.25 0.075–0.1 0.025–0.05

in BS 8004 and similar standards worldwide. For rock such as sandstone or granite with an intact compressive strength of 12.5 MPa (just break by hand), the allowable bearing pressure would also be 12.5 MPa, provided discontinuities are widely spaced apart, reducing to about 10 MPa as discontinuity spacing is about 0.5 m and reducing to 2.5 MPa when discontinuity spacing is 150 mm. If the fracturing is particularly adverse or includes discontinuities with low shear strength that could combine to form a failing wedge, then this needs specific consideration and analysis, as dealt with by Goodman (1980) and Wyllie (1999).

Variability across the foundation footprint may also be an issue. If there are soft or weathered pockets, these may need to be excavated and replaced with concrete or other suitable material. Karstic conditions with voids at depth that may be particularly difficult to investigate comprehensively can pose particular difficulties for foundation design and construction, as illustrated by a case example in Chapter 7 and discussed by Houghten & Wong (1990). Conversely, if there are particularly strong areas – for example, an igneous dyke through otherwise weak rock in a pad foundation, then this must be accounted for, otherwise the foundation may fail structurally. In all cases, it is essential to check any assumptions from preliminary design as the foundation excavation is exposed. If the ground is worse than anticipated then redesign may be required. In severe cases where, for example, a major fault is exposed unexpectedly, the required change in design may be drastic, but that is the price paid for an inadequate site investigation. Time must be allowed for checking during construction and taking any actions that prove necessary.

For soils, compressibility and settlement is often the main concern and much more so than for rock. The presumed values given in Table 6.1 should restrict settlement to less than 50mm in the long-term, but estimates may be widely in error and even supposedly sophisticated methods of prediction are often inaccurate. For foundations on granular soils, empirical methods relying on SPT or CPT data tend to be used for predicting settlement. Burland & Burbridge (1985) compiled data for sand and gravel and showed that predictions of settlement are often in error by factors of two or more. Das & Sivakugan (2007) provide an updated review.

For cohesive soil, where relatively undisturbed samples can be taken to the laboratory, oedometer tests are used to determine settlement potential and to predict rate of consolidation. Estimates of settlement can be made, given the thicknesses of the various strata in the ground profile, their compressibility and the stress changes. Details are given in many references, including Tomlinson (2001) and Bowles (1996). For major structures, engineers will often carry out numerical modelling using software such as Plaxis or FLAC, which can be used for sensitivity studies. Such software is also used to predict deformations during different stages of excavation and construction and to determine support requirements.

6.4.2 *Buoyant foundations*

If the weight of the soil removed from an excavation is the same as the building constructed within the excavation, then no settlement should occur, as illustrated schematically in Figure 6.4. This design concept has been used for many major structures incorporating deep basements which can be utilised for parking spaces. There may be a need to include holding-down piles or anchors in the design to combat any uplift forces. Construction of deep foundation boxes often involves the construction of diaphragm walls using the same techniques as for

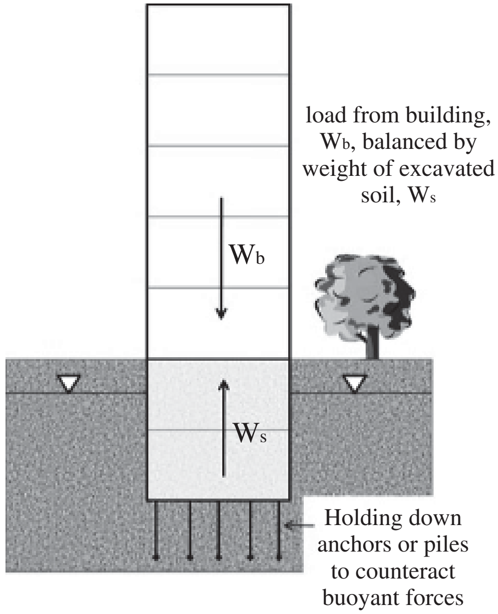


Figure 6.4 Concept of buoyant foundation design. The weight of the building balances the excavated soil so that the net increase or decrease in pressure is minimised.

barrettes, as discussed below. Once the walls are in place, excavation is conducted inside the walls, with either bracing and/or anchorages used to stabilise the works.

6.4.3 Deep foundations

6.4.3.1 Piled foundations

Piles are used to transfer building loads, via pile caps, to deeper levels in the ground profile. There are two main types: driven and bored. Driven piles are hammered into the ground and are also termed displacement piles. Hammering is sometimes done by dropping a large weight on the top of the pile from a crane or using a diesel or hydraulic machine (Figure 6.5). Bored piles are generally constructed using bucket augers, soil grabs and rock roller bits, with heavy-duty rock cutting tools used to grind their way into the underlying rock and to form rock sockets as necessary. Even using the most powerful equipment, formation of sockets can take a very long time, advancing perhaps only 100mm per hour in strong rock, and therefore can be relatively expensive, so designers should be wary of being ultra-conservative in their specification of socket length.

6.4.3.1.1 DRIVEN PILES

Driven piles are generally made of timber, steel or concrete. Figure 6.6 shows concrete piles being manufactured on site in a factory-type operation to allow 20,000 piles to be driven in just 18 months for Drax Power Station completion (Hencher & Mallard, 1989). The purpose-made pile beds were heated to allow rapid curing of concrete,

Figure 6.5 Diesel hammer (centre of photo) being used to drive pre-stressed concrete piles, Drax Power Station, UK. Elsewhere, piles are being pitched into holes formed by auger. In the background, the kentledge can be seen for a proof test on a working pile.

and the piles were pre-stressed to improve their resistance to tensile stresses and to allow the piles to be lifted from their forms quickly. For most sites, piles will be manufactured off site, sometimes as different lengths that are joined together on site to suit requirements. One of the advantages of using driven piles is that an estimate can be made of the driving resistance, given the known energy being used to drive the pile and the penetration into the ground per blow of the hammer. Piles are therefore driven to a set, which is a predefined advance rate (such as 25 mm for 10 blows by the hammer). However, resistance during driving may not always give a very good indication of how the pile will behave under working conditions, because of false sets, generally due to water pressure effects, as described for the Drax operation in Chapter 7.

Figure 6.6 Piles being cast in formers, Drax Power Station, UK. Note lifting eyes cast into the concrete piles, steel plates at end of piles (trapezoidal) and pre-stressing cables, which are to be cut before lifting piles from the casting beds.

Driving resistance and set are, however, part of the process of quality control during construction. Pile driving analysers (PDAs) using accelerometers and other instruments attached to the pile can be used to estimate driving resistance in a more sophisticated way than the traditional method of measuring the quake with a pencil, although the same limitations apply regarding whether or not dynamic behaviour is a reliable indicator of future performance. PDAs are sometimes used after the pile has been installed (both driven and bored piles) to test its capacity, but this can be somewhat of a black art with many assumptions being made and the method is certainly not foolproof or as reliable as full static load tests, as discussed below.

6.4.3.1.2 BORED PILES

Bored piles are excavated as described earlier. Temporary or permanent steel tubes (casing) may be used to prevent collapse of the hole and, if the hole is formed below the water table, often bentonite or some other mud or polymer is used to support the sides of the hole. Once the hole has been completed and cleaned out, then a steel reinforcing cage is introduced and, finally, concreting carried out. Concrete needs to be tremied by a pipe from the surface to the bottom of the hole. This avoids the concrete disaggregating, and the concrete will hopefully displace soft sediment that might have accumulated at the bottom of the bored hole after the final clean out. It will also displace the bentonite slurry or water from the bored pile excavation, so this can be a very messy operation. Despite best efforts, soft toes of sediment will still sometimes occur (perhaps associated with the removal of temporary casing) and sometimes ground movements occur causing necking of piles. Clearly, there is a need for high-quality work and for close supervision. Currently, in Hong Kong, all bored piles are installed with steel tubes attached to the reinforcing cage (Figure 6.7). After concreting, rotary drilling is carried out down one of the tubes, through the concrete and into the underlying natural ground, to prove that the pile is founded as designed and that there are no soft sediments. If there are, then remedial measures such as pressure grouting might be needed. Other tubes installed through the concrete are used to carry out geophysical cross-hole tests (seismic) to check for necking and other construction defects. In severe cases, piles may prove inadequate to carry the loads and remedial works are required. This might not be discovered until the superstructure is constructed. In one extreme case in Hong Kong, two 44-storey tower blocks had to be demolished. Such problems may be put down to workmanship, the inherent difficulties of the operation, poor investigation and design and sometimes fraud (Hencher *et al.*, 2005).

Once the piling is completed, a pile cap is constructed as a reinforced box of concrete that bridges between several piles to support major columns in the superstructure.

Figure 6.7 Reinforcing cage for bored pile with included tubes to allow proof drilling through the toes of the completed pile and cross-hole geophysical testing to prove integrity, Hong Kong.

6.4.3.2 Design

Piles are designed to suit the ground profile. If rockhead is at relatively shallow depth and the overlying soil does not contain boulders that could cause difficulties, then driven piles might be adopted, end bearing onto the rock (Figure 6.8a). At Drax, the piles were driven to found several metres into dense sand overlying sandstone, thereby picking up some skin friction as well as end bearing (Figure 6.8b). If there is no rock, then the piles will need to gain their resistance mostly from skin friction in the soil. For example, the Sutong Bridge across the Yangtze River, China, which is (in 2011) the longest cable-stayed bridge in the world, with a main span of 1,088m, is founded on bored piles taken to 117m and relying upon skin friction from alluvial sediments (Figure 6.8c).

Ways to estimate skin friction parameters and end-bearing resistance are given in textbooks such as Tomlinson (2001) and might be governed by standards such as AASHTO (2007), used as the basis for design of the 2nd Incheon crossing completed in 2009. The principles are quite simple: skin friction is calculated as soil shear strength times some adhesion factor multiplied by the surface area of the pile shaft. End bearing is often calculated as an empirical value for the soil or rock quality multiplied by the basal area of the pile. At some sites, the bottom end of the pile is enlarged by under-reaming to increase the end-bearing contribution, although sometimes the difficulty of this operation is hardly justified by the increase in pile capacity that might ensue.

A worked example of pile design to Eurocode 7, using partial factors specified uniquely for the UK (to correlate with traditional design experience), is presented in Box 6-3, based on one

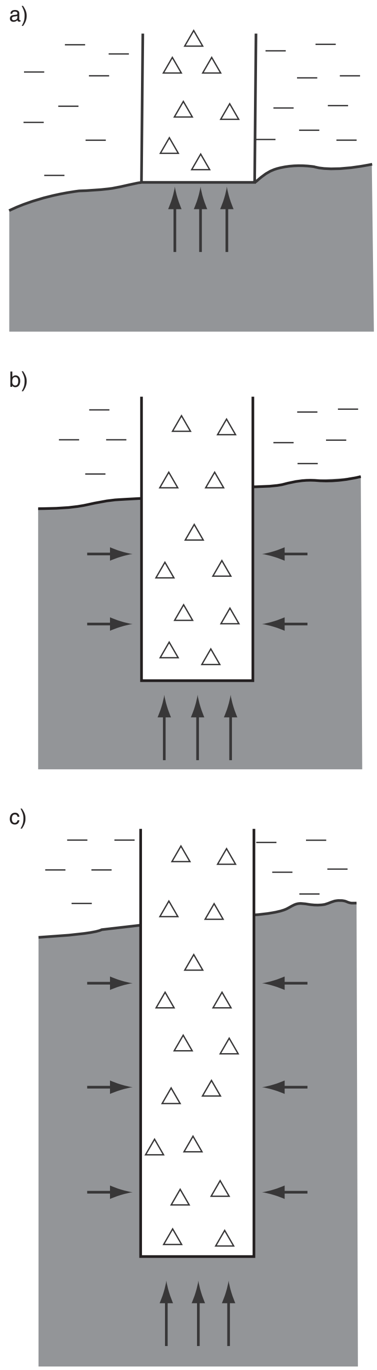


Figure 6.8 Design concepts for piles. a) End bearing, b) end bearing plus skin friction and c) skin friction dominating.

presented by Bond & Simpson (2010). Other countries might use different partial factors and other approaches, as allowed in the Eurocode. In the example presented, the main unknowns – variable live load, shaft resistance and base resistance – are factored up and down as appropriate towards a safe solution. The results

Box 6-3 Pile design (based on Bond & Simpson, 2010)

Shaft resistance, $Rs = 87.97 \times (\alpha \times 158)/\gamma$[1]

where α is an adhesion factor (take as 0.5) and γ is a model factor = 1.4

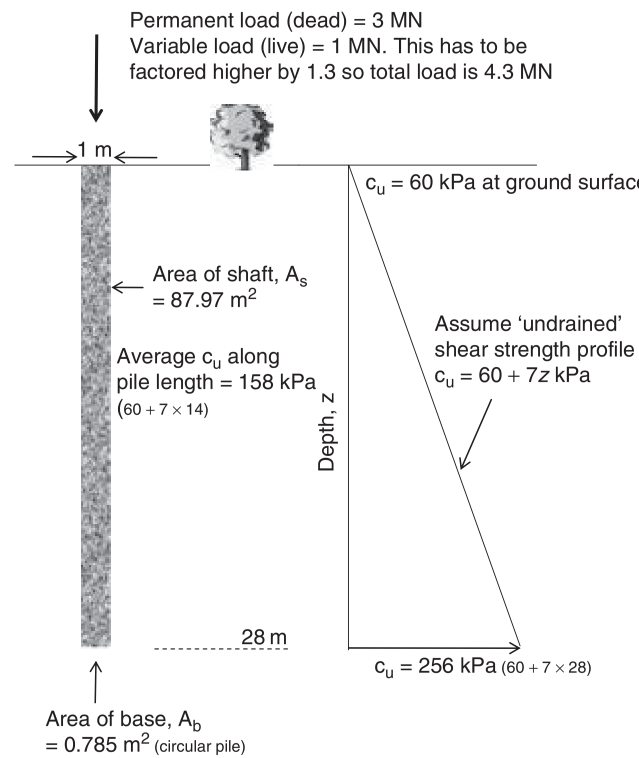
So $Rs = 4{,}964$ kN

Base resistance, $Rb = 0.785 \times (Nc \times 256)/\gamma$

where Nc is a bearing capacity factor (take as 9) and γ is the same model factor = 1.4

So $Rb = 1{,}292$ kN

Total design compressive resistance, $Rc = 4{,}964/1.6 + 1{,}292/2.0 = 3{,}749$ kN



With partial factors 1.6 on shaft and 2.0 on end bearing, which are designated in the UK National Annex to BS EN 1997–1 for a bored pile not subject to test. If tested, factors would be 1.4 and 1.7 respectively (see Bond & Simpson, 2010 for discussion).

The factored design resistance, 3.749 MN, is less than the factored compressive load, 4.3 MN, so longer piles would be required.

The traditional, unfactored global FoS would be (4,964 + 1,292) × 1.4 / 4,000, i.e. 2.19, which is lower than would normally be required following traditional design rules (typically 2.5 to 3.0) in the absence of extensive preliminary and working pile tests.

[1]Note that the symbol γ has been used in Eurocodes to represent 'partial factors'. This can lead to confusion in that the symbol γ is more generally used to represent unit weight as in γ_R for the unit weight of rock.

are compared to the FoS, as determined using a traditional approach – best estimate of strength divided by best estimate of loading. It is to be noted from this example that whichever approach, there is considerable judgement and approximation involved. Shear strength is taken as undrained, which is conceptually questionable for the long-term; adhesion factor estimates range from 0.3 to 0.9 for different soils. If an effective stress approach was adopted – as would generally be done for sand and weathered rock – then estimates would be needed of stress conditions and shaft resistance coefficients, which also requires estimation and judgement. Workmanship may also play a key role in whether or not shaft friction will be mobilised and whether the base of a bored pile excavation is properly cleaned out prior to concreting. The use of a partial factors approach does concentrate on where the key unknowns are (rather than geometry and fixed loads) but doesn't take away the need for proper ground characterisation, analysis and design judgement. The fixed nature of the partial factors might seem rather prescriptive to cover all soil, rock and founding situations. Selection of parameters, adhesion and shaft resistance factors are reviewed well in GEO (2006), and the use of Eurocode 7 for design is summarised by Bond & Simpson (2010).

A site-specific way to obtain design parameters, especially for large projects, is to install test piles and measure their performance at perhaps 2.5 times the design load of the working piles. Test piles are often instrumented along their length using strain gauges so that the actual resistance being provided by the ground can be measured throughout the full profile, and these parameters can be used in the design of other piles. Traditionally, piles are loaded from the top using kentledge of concrete blocks or steel (Figure 6.9). Jacks are used to push the pile into the ground whilst the kentledge provides the reaction. One of the difficulties of this is that much of the support comes from the upper soil at early stages of the test, and there is little idea of how the toe is

Figure 6.9 Pile test set up with kentledge. Donghai Bridge, China. Figure courtesy of Leonard Tang, Halcrow.

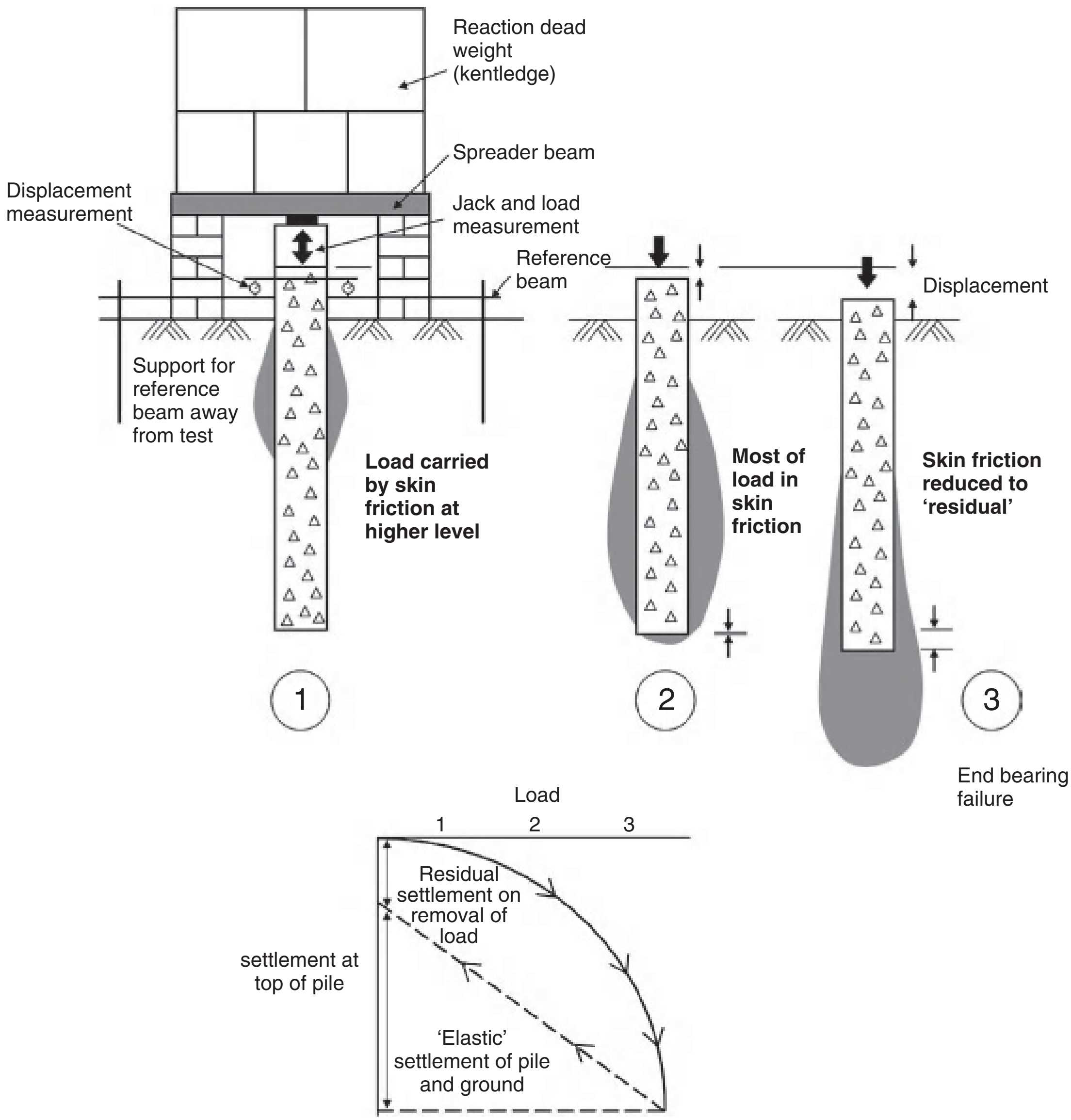


Figure 6.10 Typical set-up for pile load test. At early stages (1), most of the ground resistance will come from skin friction at shallow depths. End bearing is not mobilised until later stages (2) and (3) of the test (depending on the configuration of the pile and ground profile). The rate of settlement increases as the ground resistance becomes fully mobilised and there will be some permanent displacement (residual settlement) once the pile is unloaded.

performing until a test approaches failure (Figure 6.10). Recently, a system has been introduced where Osterberg cells are incorporated into the pile construction at depth and then expanded against the test pile, both upwards and downwards (Figure 6.11). The end-bearing resistance below the cell is balanced by the skin friction from the soil above the cell. This system was used for the Incheon Bridge design, using

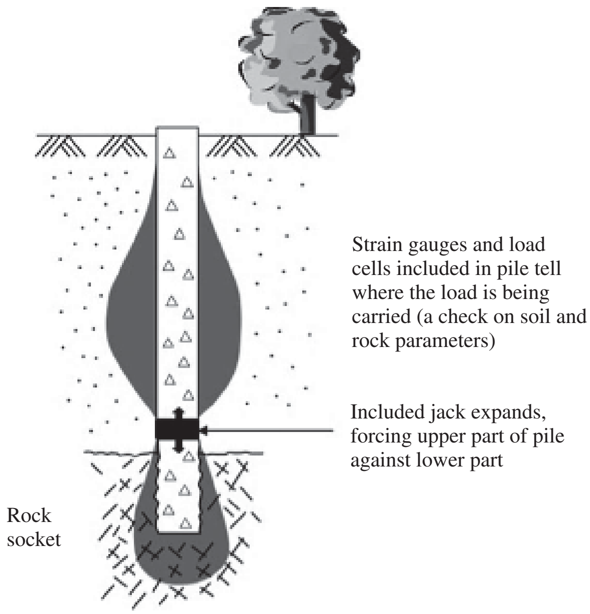


Figure 6.11 Pile test using an integral jack or set of jacks. This set-up allows the end bearing part of the pile to be jacked against the upper parts (skin friction). If strain gauges are built into the pile, then a good interpretation can be made of ground parameters.

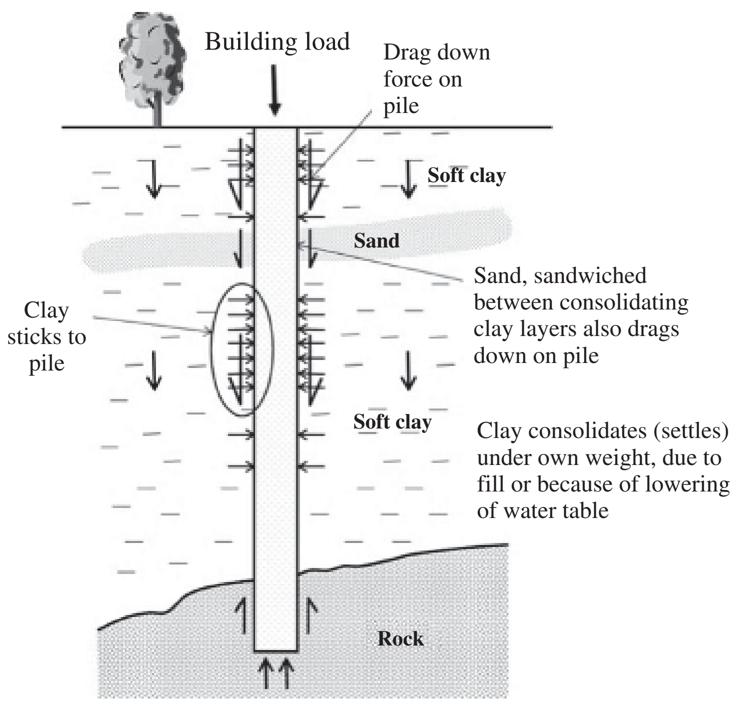


Figure 6.12 The concept of negative skin friction. Where the ground around a pile or group of piles settles significantly (cm), then the ground will cause a drag down force on the pile. At the same time, the upper parts of the pile cannot provide positive skin resistance.

up to 5 cells in a single 3m diameter pile to generate forces of over 30,000 tonnes (Cho *et al*., 2009b). The obvious advantages include the fact that no reaction is required at the ground surface, but a limitation is that the forces upwards must be balanced by those downwards, which would be difficult to achieve where the pile is mostly end bearing.

An additional aspect to be considered in pile design is possible future settlement of the ground around the pile due to self weight, earthquake liquefaction or perhaps groundwater extraction, which can result in a drag-down force on the pile, known as negative skin friction. This is illustrated in Figure 6.12. The potential for negative skin friction is generally a matter of engineering judgement based on the ground profile and perceived future usage of the site and applied as a nominal additional load to be carried by the piles.

6.4.3.3 Proof testing

Proof tests are typically carried out on one in a hundred piles or so. The test pile should be selected by the supervising engineer, after construction and with no pre-warning to the contractor so that he does not exercise special care in its construction. Full loading tests are carried out with kentledge or some other reaction system such as ground anchors and should be taken up to loads of perhaps 1.5 times the working load for the pile. The displacement during the test (partly elastic deformation of the pile) and residual settlement after the test is completed are used as criteria of whether the tested pile and its neighbours are acceptable (Figure 6.10). If not, then additional piles may need to be installed and the existing piles down-rated. There may be time or space restrictions (such tests are very expensive and time consuming) and the contractor might urge the use of dynamic pile analysers as an alternative way of proving acceptability. As noted earlier, such tests are often unreliable and may give no measure of end-bearing resistance. Specialist tests are used to determine pile integrity, for example, by using a vibrator to take the pile through a series of frequencies so that its response can be measured. Resonance will indicate the length of the responding section, which will help in deciding whether or not the pile is broken.

6.4.3.4 Barrettes

Barrettes, like piles, are deep foundations but constructed in excavated trenches using special tools called hydrofraises, often under bentonite to support the sides of the trench. Otherwise, construction is similar to a bored pile, with a steel cage inserted in to the trench prior to concreting. Barrette shapes can follow the geometry of load-bearing walls in the finished structure.

An example of the use of barrettes rather than bored piles is for the International Commerce Centre (ICC) in Hong Kong. The 118-storey building is the tallest in Hong Kong and fourth tallest in the world (in 2011). Granite bedrock is reportedly 60–130 m deep below the building, and the designers decided to use 241 post-grouted rectangular barrettes rather than more traditional end-bearing bored piles (Tam, 2010). The barrettes were cracked by high-pressure water injection down pre-installed pipes whilst the concrete was still at low strength. Once the concrete had reached its 21-day strength, high-pressure grouting was carried out through the cracked path around the barrettes, metre by metre from the base to improve the skin frictional resistance.

6.4.3.5 Caissons

Caissons are large box structures formed of steel or concrete and are used as a common solution for bridge foundations offshore. The box is typically constructed onshore then floated and towed to its location where it is sunk. Sometimes caissons are sunk into the ground by driving and digging, elsewhere they just sit on a prepared surface on the sea floor. Different types are illustrated schematically in Figure 6.13. Once the caisson is in place/sunk to the required depth, then it is backfilled with rock and concrete. Caissons are also often used to form sea walls for reclamation schemes, the boxes are formed on land then floated and towed to position where they are sunk onto prepared foundations and then backfilled.

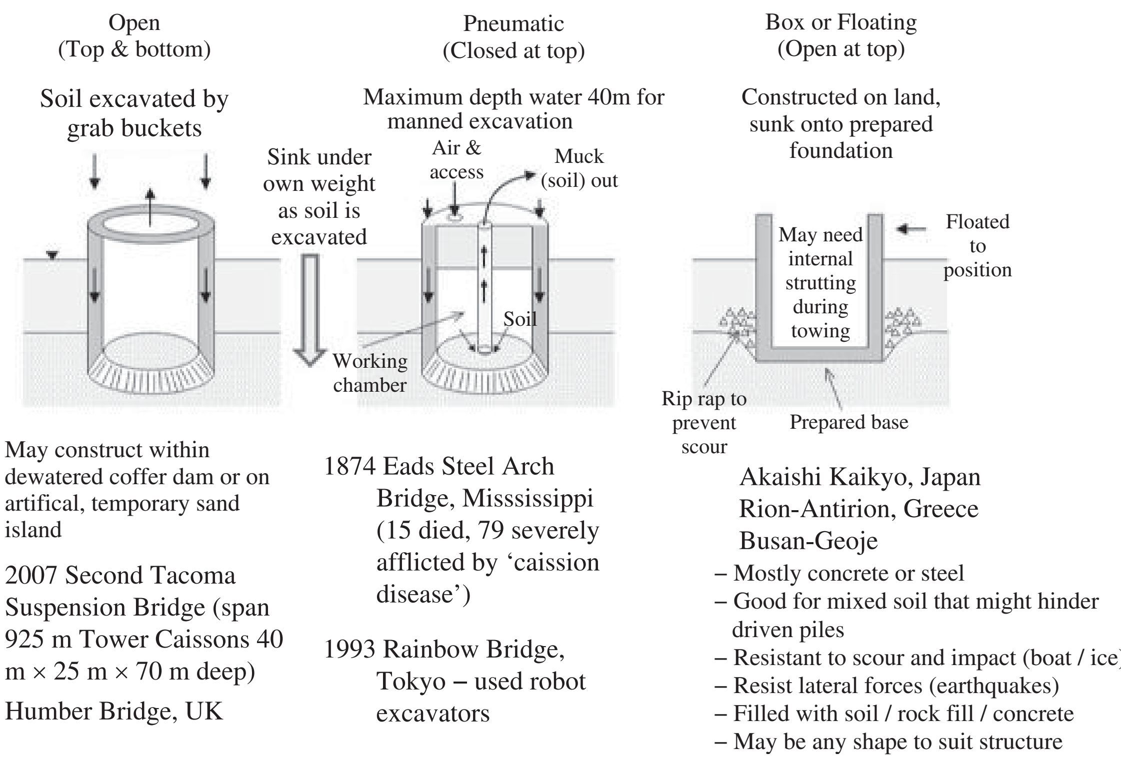


Figure 6.13 Different types of caisson commonly used for large bridge foundations, with examples.

6.5 Tunnels and caverns

6.5.1 General considerations for tunnelling

Tunnels will be constructed as part of an overall project, for example, water supply, drainage, rail, road, or in connection with power generation. As a result, there may be little flexibility over route and, therefore, geological and hydrogeological conditions and size and shape of tunnel. It is up to the engineering team to come up with a cost-effective solution.

One factor that will influence the chosen method of construction and lining (or not) are the final finish requirements for road and rail tunnels and whether or not it might carry water under pressure in hydraulic tunnels, as addressed at 6.6.5 below. The main issues for the engineering geologist and design team are likely to be:

- The geology along the route; how this will affect the selected method of tunnelling and any particular hazards such as natural caverns, mining or major faults.
- Stress levels and ratio of vertical to horizontal stress. High stress at depth and the concentrations in stress resulting from perturbation of the stress field by the construction can result in failure of the rock, which might result in spalling in brittle rocks or squeezing in generally weaker rocks (Hoek & Brown, 1980; Hoek *et al*., 1995).
- Hydrogeological conditions and the risk of unacceptable water inflows and possible flooding; this is always a major issue for undersea tunnels, but can also be a concern under land.
- Existing structures that might be adversely affected by the tunnel during construction, for example, by blast vibrations or undermining as the tunnel passes by. In the longer-term, lowering of groundwater may cause settlement and/or affect water supply boreholes.

As for all geotechnical work, one needs a ground model for design. Because tunnels are often long and may be at great depth, it may be impractical to do more than a rather superficial investigation, relying largely on geological mapping and extrapolation of data, although if a serious obstacle is anticipated, such as a major fault zone, then boreholes might be targeted at that feature using inclined boreholes or even drilled along the line of the tunnel. Alternatively, a small-diameter pilot tunnel might be constructed before the main tunnel – possibly for later use as a drainage or service tunnel – because small diameter tunnels tend to have fewer difficulties (Hoek, 2000). The pilot tunnel essentially works as a large-diameter exploratory borehole.

The ground model needs to include estimates of rock or soil quality along the tunnel drive. For rock, this is often done using rock mass classifications (RMCs) such as Q, RMR or GSI, described in Chapter 4 and Stille & Palmström (2003). This will allow some estimation of support requirements and allow a contractor to choose his method of working and type of machine if a tunnel boring machine (TBM) option is selected (Barton, 2003). The ground model will also be used for hazard and risk analysis, as discussed later, and may sometimes be used as the basis for Reference Ground Conditions in Geotechnical Baseline Reports (Chapters 2 & 4), against which any claims for unexpected or differing ground conditions can be judged. As noted in Chapter 2, however, RMCs may be too coarse to represent geological conditions realistically. They may also be open to different interpretations, so that disputes are difficult to resolve.

6.5.2 Options for construction

Up to about a century ago, all tunnels in soil or rock were excavated by hand, using explosives where necessary to break up the rock in advance of mucking out. Nowadays, many are excavated using powerful machines. The main options generally adopted in modern tunnelling and typical support measures are set out in Table 6.2. The method of tunnelling will often be decided on factors including length of tunnel, availability of TBM, local experience and expertise. In South Korea, for example, most rock tunnels, including very long ones, have been constructed in preference by drill and blast rather than TBM. There is a wide variety of tunnel boring machines designed for all kinds of conditions from rock to soft soil. The engineering geologist needs to be able to predict the ground conditions so that the tunnel designers and tendering contractors can select the correct machine. It usually takes a long time to manufacture and launch a TBM with a whole series of ancillary equipment in the following train, and if the machine proves unsuitable, for any reason, it can be a costly mistake. Some machines are designed to be able to cope with mixed ground conditions but can still run into difficulties. Nevertheless, many TBM tunnels proceed well and at much faster rates than hand dug/drill and blast tunnels. The adoption of hazard and risk analysis (BTS, 2003), as discussed at 6.5.8, will help reduce incidents but will not necessarily eliminate hazards entirely.

Table 6.2 Options for tunnelling (after Muir Wood, 2000).

Ground type	*Excavation*	*Support*
Strong rock	Drill and blast or TBM	Nil or rockbolts
Weak rock	TBM or roadheader	Rockbolts, shotcrete, etc.
Squeezing rock	Roadheader	Variety depending on conditions
Overconsolidated clay	Open-face shielded TBM or roadheader	Segmental lining or shotcrete etc.
Weak clay, silty clay	EPB closed-face machine	Segmental lining
Sands, gravel	Closed-face slurry machine	Segmental lining

6.5.3 Soft ground tunnelling

Soft ground, including severely weathered rock, may be excavated by hand or by tunnel boring machine. For open-face excavation, behaviour can be predicted using classification such as the Tunnelman's Classification of Heuer (1974), which allows prediction of whether the soil will stand firmly whilst the liner is put in place or is likely to ravel, run, flow, squeeze or swell. Behaviour depends on the nature of soil, water conditions and stress levels. For example, un-cemented sand might be expected to flow below the water table, especially at depth. Such empirical predictions are also useful for weathered rocks where the application of conventional soil mechanics principles is questionable (Shirlaw *et al*., 2000). When tunnelling in soil or in mixed-face conditions, it is the behaviour of the weakest or most mobile material that generally governs the need for, and magnitude of, the support pressure that is needed at the tunnel face.

If the soil is stiff and cohesive, then NATM methods can work successfully, as has been achieved, for example, in the London Clay (van der Berg *et al*., 2003) and in the Fort Canning Boulder Bed and the Old Alluvium in Singapore (Shirlaw *et al*., 2000). Where soils are unstable, then various options include grouting, dewatering, freezing or the use of compressed air. All of these are costly, may have severe health and safety implications and restrictions, and take time to install. Nevertheless, such methods are often necessary to recover and restart a tunnel that has encountered a major problem and perhaps collapsed.

Tunnel boring machines used in soft ground are of the closed-face type, as illustrated in Figure 6.14 a and b. Guidance on machine selection and use is given by the British Tunnelling Society (BTS, 2005).

Earth pressure balance (EPBM) and slurry machines use pressurised soil at the cutting face to hold up the ground as the tunnel advances. In an EPBM machine, the broken down soil remains in the plenum chamber behind the cutting head, balanced by pressure in the Archimedes screw, which removes the spoil under the control of the operators. In a slurry machine, which tends to be used in higher permeability soils, bentonite slurry is introduced to the plenum chamber, mixes with excavated soil, which is then removed for separation, disposal and re-use (bentonite) by pipes rather than on a muck conveyor. Permanent concrete lining is formed from precast segments, directly behind the machine, and this liner is used as a reaction to push the TBM forward. TBMs often work well for the specific conditions for which they are designed but also commonly run into problems with the machine getting stuck or running into rock that is either too hard or too soft or too wet for the type of machine (see Table 6.3). Shirlaw *et al.* (2003) report cases of settlement and collapse in Singapore, even using sophisticated EPBMs. Similarly, an EPBM machine was recently stopped by silt breaching the tunnel liner on a contract in the UK. A further example is discussed in Chapter 7. Recovery

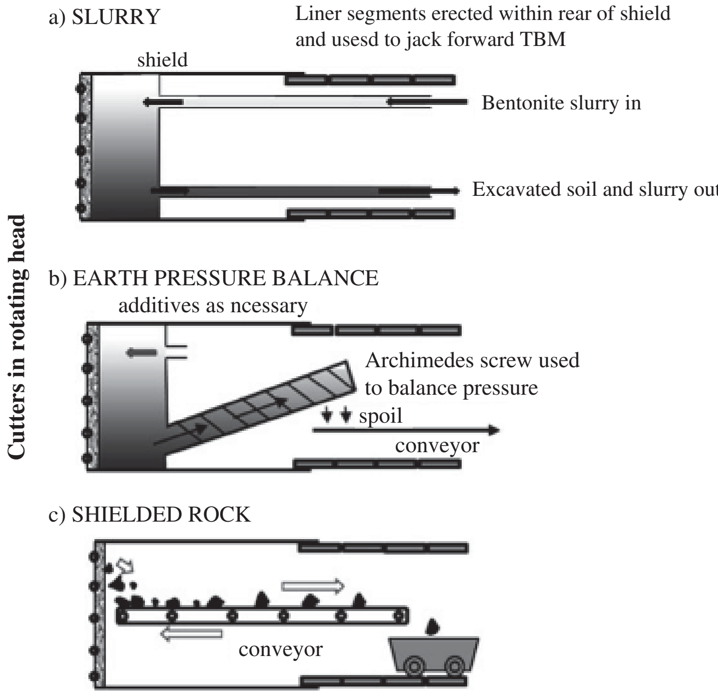


Figure 6.14 Schematic diagrams of shielded TBMs. a) Slurry machine; bentonite slurry is pumped to plenum chamber and mixes with spoil cut at the face. Mixture is removed for separation and treatment before recycling. b) Principles of EPBM. Cut soil (with additives as necessary) is removed by a screw device with the pressures monitored and maintained. c) Single-shield rock TBM. Rock cut from the face is mucked out and TBM pushes forward against the liner erected to the rear of the shield. Other rock TBMs use grippers pushed against the walls of the tunnel and use this as the reaction force for advancing the TBM.

options include freezing the ground and grouting the ground to stabilise it to allow the TBM to be withdrawn (NCE, 19 January 2011).

Where the materials to be excavated include strong and weaker material, this is known as mixed-face conditions. For stability, the major issue concerns relative mobility of the materials rather than just strength. A mixed face of strong boulders and hard clay presents problems in terms of rate of excavation, but generally not in terms of heading stability. However, a combination of strong, stable rock with a more mobile material, such as flowing, rapidly squeezing or fast ravelling material, provides conditions where the overall stability of the heading can be very difficult to control as well as difficult to excavate. Shirlaw *et al.* (2003) provide examples of major inflows resulting from the use of conventional rock tunnelling methods too close to the transition from rock-like to soil-like conditions. Ironically, this particular type of mixed-face condition has become even more problematic with the introduction of modern tunnelling technology.

Table 6.3 Typical problems with TBM tunnels and possible mitigation measures.

Problem	**Mitigation**
Ground too strong (intact strength and/or lack of discontinuities)	May need to pull TBM back and advance with drill and blast
Ground too weak and collapsing (should have been an earth balance or slurry machine perhaps)	Ground improvement might be necessary in advance of tunnel drive – grouting or freezing
Major faults	Collapse of ground and TBM gets stuck. May need to sink a shaft in front of machine and construct a tunnel back to and around the TBM to free it up. Ground treatment and possible hand construction through fault zone may be required to get the TBM going again
Weak ground and high *in situ* stresses leading to squeezing action on TBM	Can cause huge delays. Ground improvement to strengthen the ground and resist the squeezing pressures
Too much clay for slurry treatment	Can cause delay and necessitate installation of additional treatment plant – extra hydrocyclones, etc
Ground abrasive because of high silica content causing too much wear on teeth, leading to cost and delay	Cost may be prohibitive, necessitating a change of excavation method
Too much water and TBM electrics not protected	Drilling and grouting in advance of machine or possibly ground freezing or compressed air working. Possible change of method to drill and blast or employ different machine with suitable spec
Excess tunnel slurry pressure causes blowout at ground surface	Lower pressure
Pressure too low causes face collapse	Reverse of the above

6.5.4 Hard rock tunnelling

The main options are drill and blast, a roadheader excavating machine or to use a TBM that may be either open (without a protective shield) or shielded.

6.5.4.1 Drill and blast/roadheaders

Generally, drill and blast tunnels are more flexible than TBMs and allow difficult ground conditions to be understood and overcome, but they may be much more time consuming unless a number of access points can be found to allow operations to proceed from several faces at the same time.

Holes are drilled in the face, and explosives placed in the holes. Issues of tunnel blast design are addressed by Zare & Bruland (2006). The holes are detonated sequentially to break to a free face over micro seconds. The aim is to break the rock to manageable size so it can be excavated (mucked out) readily with machines, without further blasting or hammering. Other aims may be to keep blast vibrations to a minimum and not cause damage or offence to nearby residents, and usually to keep as closely as possible to the excavation shape prescribed by the designers, i.e. minimising overbreak. Typical advances per round are 3 to 3.5 m, sometimes up to 5 m in very good rock conditions. Depending on the size of tunnel and ground conditions, the full face may be blasted in one round or may be taken out as a series of smaller headings – top, or side, that may be supported by sprayed concrete with steel mesh or steel/carbon fibres, rock bolts, and/or steel arches or lattice girders, before the tunnel is advanced. Figure 6.15 shows a tunnel portal following the first blast, with steel arches being erected to protect the tunnel access.

Figure 6.15 After first blast and mucking out, construction of temporary steel arches to protect tunnel portal, Queens Valley Reservoir, Jersey, UK.

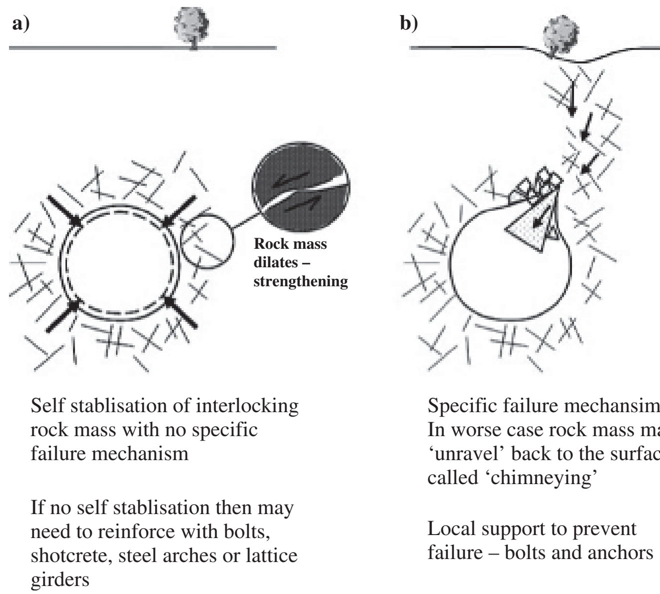


Figure 6.16 (a) Convergence in rock tunnel to stable condition. (b) Local failure and ravelling to ground surface.

After blasting, and dust and gases have dissipated and safety checks made (e.g. for methane or radon), the broken rock is mucked out and it is the engineering geologist's task to examine and map the geological conditions exposed. The freshly blasted rock may well be unstable, and the geologist should not approach the face until the contractor has carried out all necessary scaling and/or rock support work to make the tunnel safe. The contractor has overall responsibility for site safety and his instructions should be followed at all times in this respect. A decision will then be taken on whether the ground is as expected, if the ground is changing (and probing ahead is required), and the support requirements. Any potential for deteriorating conditions or, for example, a major potential wedge failure, need to be identified quickly so that support measures can be taken. As illustrated in Figure 6.16, often the rock mass is self-supporting. As the tunnel is excavated, the tunnel walls move inwards, the rock mass dilates and generally locks up. If there is an inherent weakness, such as a free wedge of rock or a fault zone, then local collapse can be followed by ravelling failure, which could chimney to the ground surface. In two of the examples discussed in Chapter 7, the situation deteriorated quickly. If conditions are poor and getting worse, then the ground might be supported in advance of the tunnel by an umbrella of spiles or canopy tubes, and/or by pressure grouting.

In suitable rock, other mining approaches may be used, including the use of large roadheaders that cut their way into the rock but do not excavate the full face profile in one operation, unlike a TBM. In a tunnel formed by drill and blast or roadheader, it is possible to examine and record the ground conditions throughout construction and make decisions as to the support required. In a TBM tunnel, little can be told about the ground ahead of the machine without stopping and drilling in front of the face, which disrupts operations and is therefore to be avoided.

6.5.4.2 TBM tunnels in rock

The design and use of modern hard rock TBMs is covered comprehensively by Maidl *et al.* (2008). In good rock with high RQD, open TBMs are sometimes used, but generally only for relatively small diameter tunnels. The tunnel advances by jacking forward against grippers that are extended laterally against the tunnel walls. Clearly, if the rock becomes poor quality then there may be problems with the grippers. There is also no way of preventing groundwater ingress other than by grouting, preferably in advance of the machine. In Chapter 7, a case (SSDS) is presented where open-rock TBMs were selected, anticipating good rock conditions with low water inflows, and the operations were halted when inflows became too great and grouting in advance was extremely difficult.

In poorer-quality rock, generally, shielded TBMs are used. A single-shield machine pushes against the liner, as for soil TBMs (Figure 6.14c). In other set-ups there are two shields; the rear shield has grippers and provides the reaction against which the front shield can push forward. The cutter head has discs that rotate as the cutter head itself rotates. The thrust of the machine causes the rock to fail, mainly in tension. A major consideration is the lifetime of the cutting discs before they need to be replaced, as addressed by Maidl *et al.* (2008). A case example in Chapter 7 describes considerable wear in an EPBM used to tunnel through abrasive sandstone.

6.5.5 Tunnel support

6.5.5.1 Temporary works

Rock tunnelling, in general, relies largely on the rock mass locking up as joints and interlocking blocks of rock interact and dilate during the process of convergence towards the excavation. Good-quality rock often forms a natural arch and no or little support is needed. However, in weaker ground, such as in fault zones, the rock mass cannot support itself, even with reinforcement, and requires artificial support in the form of steel arch ribs, typically encased in shotcrete. Optimising support requirements in weaker ground requires prediction of likely convergence rates, making observations as excavation is undertaken, i.e. observational methods, and then applying support such as rock bolts and/or shotcrete and/or steel arch ribs to control the movement and prevent excessive loosening (Powderham, 1994). In stronger, blocky rock masses, rock movement will be much less, and the purpose of the support is then to prevent loss of loose blocks and wedges, which would destabilise the arch and maybe lead to ravelling failure.

Rock mass classification systems introduced in Chapter 4 are linked to charts allowing decisions to be taken as to the immediate (temporary) support measures required. These are reviewed by Hoek *et al.* (1995). In practice, decisions may often be biased by other considerations such as the materials and equipment at hand and the workers' perceptions of the degree of risk and how well previous support measures have worked. This may of course have cost implications and may also later become a matter of dispute as to what was really necessary, as discussed and analysed by Tarkoy (1991). The importance of good engineering geological records during construction is emphasised. In severe situations such as high stress or intense water inflow, steel lining may be used but even then this sometimes proves inadequate as happened during the construction of the Tai Po to Butterfly Valley water supply tunnel in Hong Kong, where unexpectedly high water pressures buckled the liners (Robertshaw & Tam, 1999; Buckingham, 2003).

6.5.5.2 Permanent design

There are two main areas for consideration: firstly, the area around the portal, especially for tunnels that are part of a road or rail system, and, secondly, need for a permanent liner.

6.5.5.2.1 PORTAL DESIGN

The area above the entrance to a tunnel often requires careful engineering to make it safe, both during construction and during operation. The problems are essentially the same as for general slope stability design, as discussed later in this chapter, but the need for long-term inspection and maintenance, whilst maintaining tunnel usage, sets portal design in a rather special category. A canopy is often constructed to protect the portal area from falling rock and other debris, as illustrated in Figure 6.17. Catch nets, barriers (such as gabion walls) and *in situ* stabilisation can

Figure 6.17
Canopy extending out from tunnel liner (being waterproofed), to protect portal area. A55, North Wales.

be used to prevent debris impacting the portal area. Rock and soil masses immediately above the portal area are often covered with steel mesh and shotcrete or similar hard covering and dowelled, nailed or anchored back using post-tensioned bolts and cable anchors. The requirements for designing, protecting and maintaining ground anchorages are set out in national standards and codes of practice such as BS 8081 (BSI, 1989) and BS EN 1537 (BSI, 2000). Despite such standards, things occasionally go wrong, either because of ground conditions or flaws in the anchorage itself, and designers must appreciate the practical difficulties that might be associated with maintenance programmes whilst ensuring safety for the road user. If a major problem is found, then the tunnel might need to be closed or restricted in use whilst the problems are rectified. Several cases of the failure of rock anchorages, even in projects post-dating BS 8081, are discussed in Chapter 7.

6.5.5.2.2 PERMANENT LINERS

The options for permanent tunnel liner design include:

- Unlined (ignoring temporary support measures)
- Unreinforced concrete
- Reinforced concrete
- Steel.

Lined tunnels can be designed to be undrained, in which case the permanent lining must withstand the full groundwater pressure as well as rock loads. Other tunnels are designed to be drained, whereby the outer surface (or extrados) of the arch of the liner is lined with a waterproofing membrane laid onto geotextile sheets, which carry water down to drains and sumps below the tunnel invert. The sumps may need continual pumping, and the whole drainage system needs maintenance over the life of the project. Figure 6.18 shows details of a design, as used in some recent rail tunnels in Hong Kong. After shotcreting the tunnel walls, layers of geotextile (outer) and waterproof membrane (inner) are placed, followed by an inner concrete liner (250 mm thick). Groundwater is thereby channelled via the geotextile to an egg box drainage system in the invert. For any drained lining design, care must be taken that any permanent drawdown in the water table has no adverse affects on structures above the tunnel or on water supply from groundwater sources.

Precast concrete segments are commonly erected as part of a TBM excavation and support process, mainly in soft ground tunnels, but also in some hard rock applications. The segments are manufactured externally and then erected within the shield surrounding the advancing machine and bolted together. If required, segments can be fitted with gaskets to form fully waterproof concrete liners (Figure 6.19). As noted earlier, the installed liner can be designed to provide a reaction to push the TBM forward.

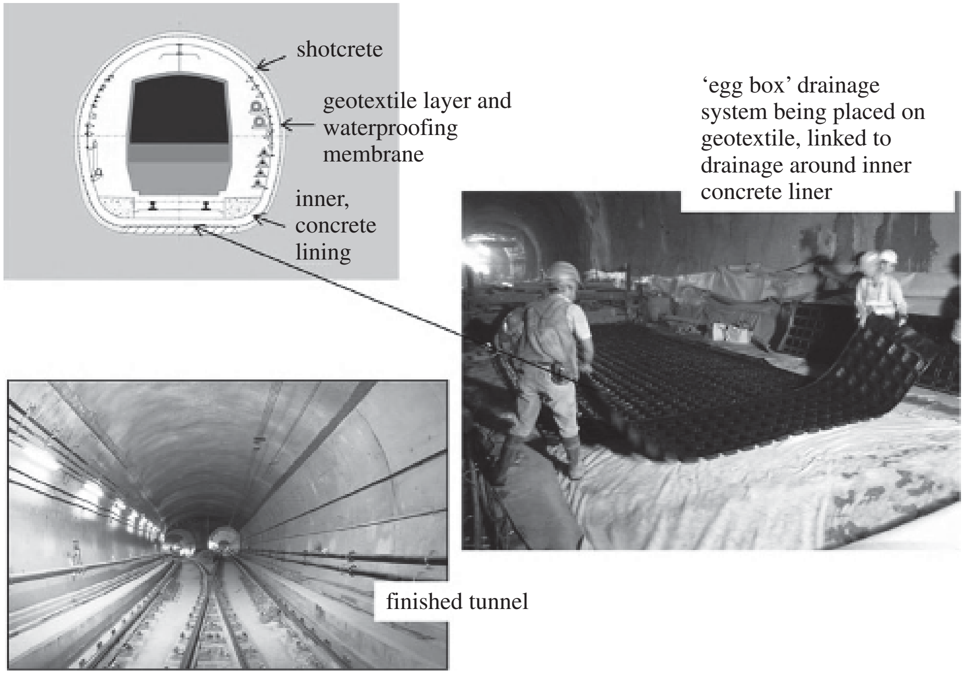


Figure 6.18 Egg box drainage system for a drained tunnel (courtesy of MTRC, Hong Kong).

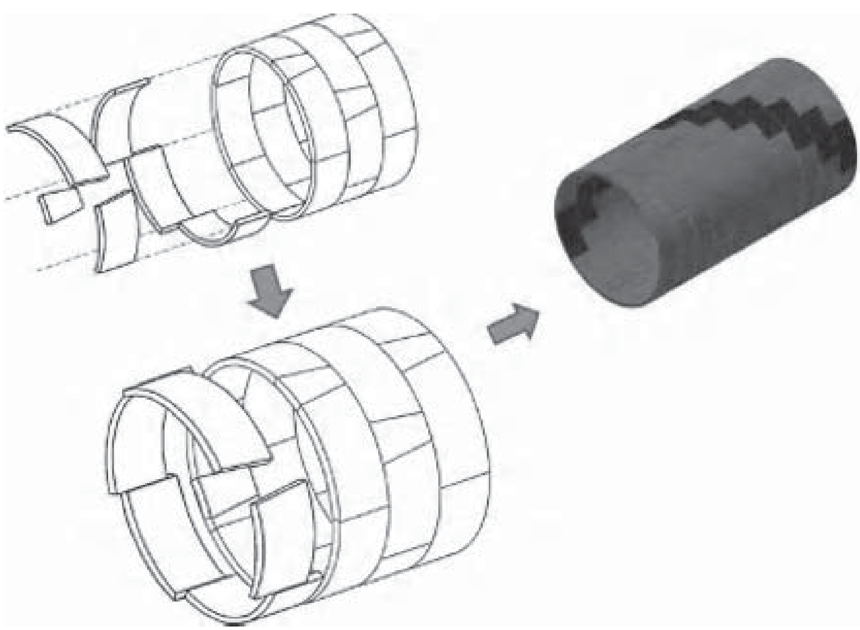


Figure 6.19 Interlocking tunnel segments, prefabricated and erected to the rear of a TBM shield (figure courtesy of Mike King, Halcrow).

One of the most severe design situations is in high-pressure water supply tunnels associated with hydropower constructions where for some operational periods the tunnel carries water under high pressure, but at other times the same tunnels are empty and have to withstand significant external water and rock pressures.

The main concerns with pressure tunnels are:

- Potential damage by hydraulic fracturing (formation of new fractures) or jacking (opening of existing fractures) within the rock mass, and
- Stability, durability and low maintenance.

To avoid hydraulic fracturing, an empirical rule is sometimes used:

$$\frac{D\gamma_R}{H\gamma_W} > 1.25 \qquad \text{(Haimson, 1992)}$$

Where γ_R is unit weight of rock and γ_W is unit weight of water, D is rock overburden at tunnel location and H is the water head. However, it is important to recognise that this formula only considers vertical *in situ* stress. Horizontal stress can be very low in some situations, for example, close to valley sides, and this will control the risk of hydraulic fracture or jacking if water from the tunnel can reach the excavated rock surface at sufficiently high pressure.

Where the confining rock stress, vertical and/or horizontal, is too low, fully welded continuous steel liners are generally used to prevent the high-pressure water from reaching the rock mass. Concrete liners may be used in competent rock but might crack under high internal water pressure if the confining stresses are too low. In such cases, there is a risk of leakage to surrounding ground (with a risk of causing landslides in some situations) and/or water flow into other underground openings. Haimson (1992) presents examples of schemes where the importance of stress conditions and the correct choice of lining only became evident late in the design process, with 'unpleasant consequences'. An important task of the engineering geologist is to ensure that the *in situ* stress conditions along the route of a pressure tunnel are evaluated fully and reported to the design team, preferably at an early stage in project planning.

In certain situations, typically in low pressure headrace tunnels, a concrete liner can be designed with drainage holes to relieve water pressure on the tunnel lining. Consolidation grouting is usually carried out around the tunnel to reduce leakage out of or into the tunnel (depending on the relative internal and external water pressures). Unlined tunnels can be used in good rock conditions and with favourable *in situ* stresses, but there may be higher maintenance requirements and the need to construct rock traps to catch any fallen debris. The proper design of hydraulic pressure tunnels is particularly important as the consequences of failure are usually very severe and costly to repair. A comprehensive summary of the principal design and construction considerations is presented by Benson (1989).

6.5.6 Cavern design

Caverns are large-span underground openings and these are used for many purposes, including sports halls, power stations and oil and gas storage (Sterling, 1993). Hydroelectric power caverns and large three-lane road tunnels are typically 20 to 25 m span, but caverns have been constructed successfully in good-quality rock with spans in excess of 60 m (Broch *et al*., 1996), and natural caves are found with much larger spans.

There is considerable guidance in the literature on approaches to their design and construction (e.g. Hoek & Brown, 1980; GEO, 1992). Many design issues are similar to tunnels but because they are at fixed locations, ground investigation decisions are more straightforward. The other major difference is scale. Whereas many tunnel walls lock up as the rock dilates, and need little support to ensure stability, in a cavern there is more potential for large-scale strain and failure mechanisms to develop. For example, large caverns were required for a proposed high-speed rail station at Taegu, Korea, in strong mudstone. Preliminary numerical analyses were carried out to design permanent concrete liners and bolting support, assuming essentially isotropic rock mass parameters. The design had to be revised when it was realised that the rock structure was strongly anisotropic with bedding mostly horizontal and many near-vertical tensile joints infilled with calcite (Figure 6.20). These joints could allow discrete failure into the crown of the openings, as illustrated by Maury (1993) for mine workings.

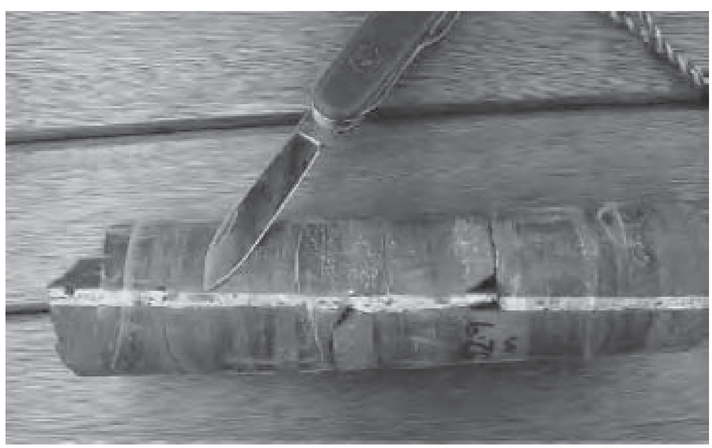


Figure 6.20 Rock core from vertical borehole in strong mudstone, Taegu, South Korea. Note near-vertical persistent joint infilled with calcite. This network of joints (two sets orthogonal to bedding) were encountered frequently in preliminary boreholes, and appreciation of their significance led to a) reconsideration of the potential rock loads on the permanent liners and b) additional ground investigation using inclined rather than vertical holes to characterise the rock mass better.

Hoek & Moy (1993) and Cheng & Liu (1993) describe different aspects of the design and construction of the Mingtan pumped storage project in Taiwan and illustrate the need for an integrated approach of geological investigation, modelling, design, observation, construction and instrumentation. An exploration/drainage gallery and two other galleries were used to install long corrosion-protected permanent cable anchors to reinforce the roof arch of the main cavern 10 m below, prior to its excavation. Small loads were applied to the cable anchors, which only took on their full loads as the cavern was excavated.

6.5.7 Underground mining

Underground mining is quite different from the formation of caverns and tunnels for civil engineering, although many of the skills required are the same. In mining, the objective is to extract the ore whilst minimising waste rock production. Safety is a prime concern, as it is for civil operations, but mining involves the formation of non-permanent voids, many of which will be allowed to collapse or packed loosely with waste rock, so the fundamental operational concepts are obviously quite different. Rock mechanics of underground mining operations are discussed by Brady & Brown (2004). In terms of geological hazards, of particular concern are flammable and/or noxious gases, including radon, and the control of dust and ventilation is very important. Such matters are generally mandated by national standards on health and safety but still accidents occur regularly worldwide.

A general concern for construction in mining areas is continuing ground settlement or sudden collapse of old workings. These are matters to be considered at the desk study stage of site investigation, as addressed in Chapter 4.

6.5.8 Risk assessments for tunnelling and underground works

In Chapter 4, a system was introduced whereby site investigation is conducted or reviewed following a checklist approach whereby firstly geological hazards are considered, then environmental factors and finally hazards associated with the specific type of project or construction method. Tunnels are often particularly risky undertakings because they are so dependent upon geotechnical conditions, which may vary considerably along their length, and it is seldom feasible to carry out as comprehensive a ground investigation as it is for other types of project. Good reviews of tunnel collapse mechanisms and case histories are given by Maury (1993) and GEO (2009) respectively. Consequently, industry has developed several approaches whereby hazards are considered in detail, so that strategies can be prepared to reduce or mitigate the risks. This can be done at the option assessment and design stages and then later as part of the management of construction.

The British Tunnelling Society Joint Code of Practice for Risk Management of Tunnel Works in the UK (BTS, 2003) was prepared jointly by the Association of British Insurers and the BTS and sets out requirements regarding risk assessment and management for any tunnel with a contract price of more than £1 million. In effect, it is mandatory in the sense that without its adoption, no insurance will be forthcoming for an underground project. The Code of Practice sets out how and when risk is to be assessed and managed, and by whom. Risks are to be assessed at the project development stage (design), by the contractor at tender stage and during construction through a risk register.

The Code also requires the ground reference conditions or geotechnical baseline conditions to form part of the contract, but as noted in Chapters 2 and 4, definition of such conditions is not always straightforward. Whilst the intention to avoid dispute is laudable, there may be considerable difficulty in summarising geological and geotechnical conditions succinctly and unambiguously.

6.5.8.1 Assessment at the design stage

The ways that risk can be assessed at investigation and design stages are illustrated by the example of the 16.2 km Young Dong rock tunnel in Korea, as presented in Appendix E-1 and E-2. Given an appreciation of the ground conditions along the route, based on a well-conducted site investigation, the hazards associated with the various options for construction can be considered. Once these have been identified, their likelihood and seriousness can be rated in terms of potential consequence (e.g. programme, cost, health and safety) and methodologies devised for mitigation prior or during construction. Decisions can then be made on how to proceed.

6.5.8.2 Risk registers during construction

During construction, hazards that were anticipated at the design stage may prove real or illusory. New ones will be identified and need to be dealt with. The current way of so-doing is to employ a risk register in which hazards are identified and assigned to individuals in the project team to derive strategies for their avoidance or mitigation. In the BTS Code of Practice (2003), this is identified as a task for the contractor but the register will include risks brought forward from the project development stage. In practice, it may well be the project engineer rather than the contractor who manages the construction risk register, perhaps at monthly meetings held to monitor progress on mitigating each of the identified risks, remove from the register those that have been dealt with, and recognise and assign to individuals any new risks identified during the course of the work. Brown (1999) outlines the risk management procedures adopted for the successfully completed Channel Tunnel Rail Link Project in the United Kingdom, and a list of typical tunnelling hazards to be considered during construction is presented in a table in Appendix E-3.

6.6 Slopes

Landslides cause major economic damage and kill many people each year. Slopes can be split into natural and man-made. The hazards from natural terrain landslides in mountainous regions and at the coast are considered in Chapter 4.

Man-made slopes include cut slopes (cut into the natural hillside) and fill slopes. Fill slopes might simply comprise the excess debris from an adjacent cutting, dumped or compacted onto the adjacent hillside to form an extra carriageway, but can also include sophisticated, high and steep slopes incorporating geotextiles or other materials to strengthen the soil (Figure 6.21). Stability needs to be assessed by engineers and if considered unstable, measures must be taken to improve the stability to an acceptable level.

6.6.1 Rock slopes

Rock slope stability is generally controlled by the geometry of pre-existing, adversely oriented discontinuities, including bedding planes, faults and master joints including sheeting joints. Failure types can be grouped as illustrated in Figure 6.22 and as follows:

Shallow: superficial failures, generally low to medium volume.

Structurally controlled: sliding may occur on one or more intersecting discontinuities that are adversely oriented relative to the slope geometry. Toppling can result because of the presence of unstable columns

Figure 6.21 Construction of reinforced earth embankment for Castle Peak Road widening, Hong Kong.

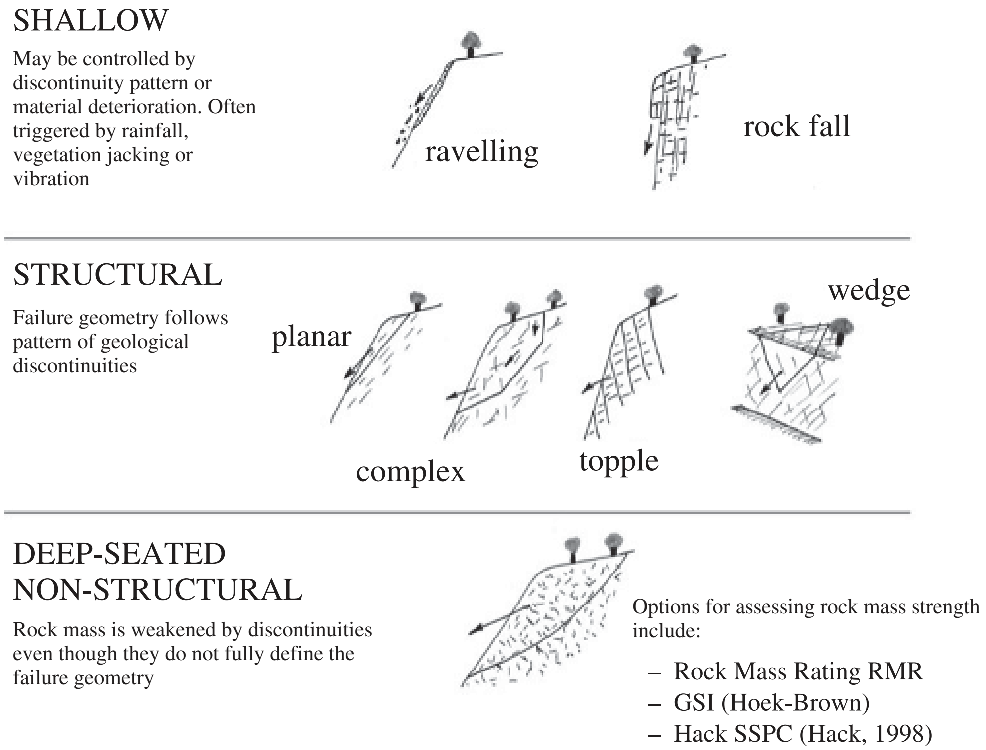


Figure 6.22 Modes of failure in rock slopes.

of rock, perhaps dipping steeply back into the slope. Large complex failures can involve a number of adverse sets of discontinuities together with some breaking through intact, perhaps weathered rock, allowing the full mechanism to develop.

Deep seated, non-structurally controlled: the rock can be considered an interlocking mass of rock blocks without adverse fabric such as bedding, schistosity or systematic joints.

6.6.1.1 Shallow failures

Steep rock slopes are sources of rockfalls, which can be a major risk, especially where adjacent to a busy road or railway. All rock slope surfaces deteriorate with time (Nicholson *et al.*, 2000). Rock material weathers, vegetation grows and opens up joints and blocks get undermined by erosion (Figure 3.58). Even small blocks can cause accidents. On large lengths of highway through a mountainous region, there will be a need to identify where the risk is greatest so that the risks can be mitigated cost-effectively (Box 6-4). This can be done by using some Rock Mass Rating appraisal system together with software capable of predicting where falling rock might end up, but it is often just a matter of engineering judgement taking account of the history of rockfalls. In such an assessment, it should be remembered that relatively minor rockfalls may be precursors to major rock collapses. Methods of mitigating rockfalls and other potential landslides are discussed below.

Box 6-4 Judging the severity of rockfall hazards and the associated risks

> 'People – even experts – rarely assess their uncertainty to be as large as it usually turns out to be.'
>
> Baecher & Christian (2003).

The assessment of hazard of rock slope failure is always rather subjective, as illustrated by a visit to the petroglyphs at Anhwa-ri, Goryeong, Korea, in February 2008. The rock exposure shown in Figure B6-4.1, above the rock carvings, appears to be on the brink of failure and one would be tempted to

Figure B6-4.1 Petroglyphs at Anhwa-ri, Goryeong, Korea. Rock slope above petroglyphs (with small protective fence) shows signs of vegetation wedging, with loose blocks resting against trees.

fence off the area, immediately followed by removal of any blocks that cannot be stabilised by dowelling and dentition works. However, the fact that the precarious open-jointed rock is directly above the ancient rock carvings, is evidence that this rock face has not retreated very far over a period of more than 2,000 years. The process of deterioration and collapse is actually quite slow and judgment of the risk as immediate and obvious, requiring urgent action, would therefore err on the conservative side.

Conversely, the slope shown in Figure B6-4.2 is in the Cow and Calf Quarry at Ilkley, Yorkshire, in the UK, and was used to teach MSc engineering geology students to map rock discontinuities for several years. The collapse to the left of the photograph occurred unexpectedly between mapping exercises, despite its repeated examination and systematic logging on scan lines, without the failure mechanism having been identified.

Figure B6-4.2 Unexpected rock failure in Cow and Calf Quarry, Ilkley, West Yorkshire, UK.

These examples illustrate our uncertainty and the difficulties in judging the degree of hazard by examination alone. It is highly likely that even after ground investigation, our ability to judge the severity of the situation is often rather poor. The conclusion must be that consequence should be the priority when assessing the risk of slope failure. If there is a major risk to life, then works should be done. This is the underlying philosophy behind the Landslide Preventive Works (LPM) strategy in Hong Kong where the catalogue of tens of thousands of slopes, prepared in the 70s and 80s, has been compiled and ordered in terms of perceived risk (a function of height, angle and proximity to vulnerable facilities). Each slope is being checked and upgraded in order. Most of these are dealt with using essentially prescriptive engineering works, including soil nails and inclined drains installed to a pattern.

If there is clear danger from the hazard, then it should be dealt with. In the Korean case discussed above, despite the apparently slow retreat of the rock exposure above the petroglyphs, visitors to the site should be protected against the evident rockfall hazards.

Quantitative risk assessment of rockfall to roads

At the site shown in Figure B6-4.3, while it might be intuitively obvious that there is some risk to life from rockfall along the road and some history of such rockfalls, the cost of preventive works may be very expensive. One way to deal with this quandary is to try to quantify the risk and compare this to the cost of reducing the risk.

To do this requires the following data to be measured or estimated:

– Frequency and size of rockfall incidents (per day).
– Number of vehicles per day, average length and velocity.
– Vulnerability of persons in vehicles to rockfall (depends on size of falls).

Figure B6-4.3 Road cut through limestone with very little engineering support or protective measures, Tailuko Gorge, Taiwan.

The annual probability of risk of death can then be calculated and compared to published guidelines on acceptable risk (e.g. Fell *et al*., 2005). Different sections of road will be shown to have different risk levels, which will allow decisions to be made on where to carry out mitigation works. Quite often such a calculation will show that risks are acceptable even if, judgmentally, the hazard is still intolerable (the situation looks very worrying). It may well be found that relatively simple measures, such as scaling off the most obvious loose rock and providing netting or cheap barriers such as gabions locally, will reduce risk considerably whilst also making the situation feel safer. Further guidance on judging rockfall hazards and the use of rockfall rating systems is given by Bunce *et al*. (1997) and Li *et al*. (2009).

6.6.1.2 Structural

The distinction of failure mechanisms into planar, wedge and toppling, and the discontinuity geometries and conditions responsible for each style of failure, are set out clearly by Hoek & Bray (1974), and this has been updated by Wyllie & Mah (2004). The most common type of failure is sliding on a single discontinuity, and this is simple to analyse. The main difficulties are in assessing shear strength of the rock discontinuities, as set out in Chapter 5, and how to deal with groundwater pressures. Generally, a simple analysis is done in which it is supposed that water pressure at the slope face is zero, increasing back within the slope, to some height below ground surface at the rear of the slope (Figure 6.23). This is often a conservative assumption, in that water pressure will be localised, not acting throughout the whole slope at the same time. Richards & Cowland (1986) discuss a well-investigated site where it would have been unrealistic to design the slope to withstand the maximum water

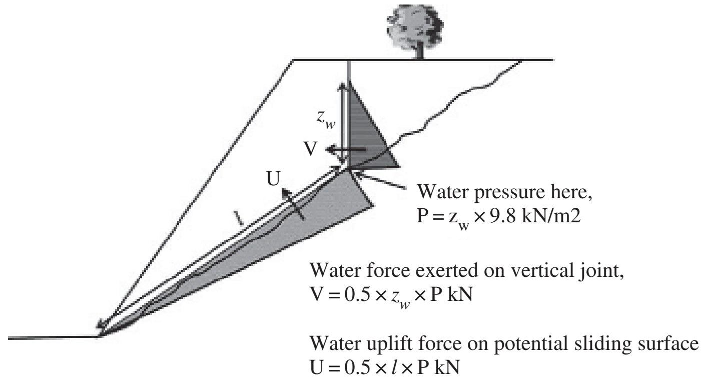


Figure 6.23
Typical model for analysing influence of water pressure on stability of a sliding rock slab.

pressures at each location, all acting at the same time, because instruments clearly showed pulses of water pressure travelling through the slope, following a rain storm.

Even small intact rock bridges can provide sufficient true cohesion to stop seemingly hazardous slopes from failing (Figure 5.19). This can be a major dilemma because the rock bridges cannot be seen or identified by any realistic investigation method. Careful geological study has failed to identify a useable link between persistence and any other measurable joint characteristic (Rawnsley, 1990) and, it must be remembered, traces exposed at the Earth's surface may be poor representations of characteristics inside the unexposed mass, because of stress relief and weathering. Because of this uncertainty, designs will typically require the risk of failure to be minimised by incorporating toe buttresses, reinforcement with anchorages of some kind, or some other protection, possibly using an avalanche shelter.

From experience, wedge failures are relatively rare so that even where these are identified as a problem from stereographic analysis, this might not develop in practice. Similarly, most slopes that appear to have a toppling problem do not do so in reality, generally because of impersistence. Care must be taken, therefore, to be realistic in appraising the results of any geometrical analysis that suggests there to be a problem. One factor that must be considered is risk, which is the product of hazard (likelihood of a failure) and consequence (likelihood of injury or damage). One other aspect is that where major failures do occur, it is often found by later inspection that the rock mass was in serious distress long before failing and this might have been discovered by carefully targeted investigation. Key factors to look for are open and infilled joints and distorted trees, though again the situation might be less risky than it immediately appears (Box 6-4). There is no easy answer to this – it is a matter requiring observation, measurement, analysis, experience and judgement, and consideration of consequence.

Monitoring can be conducted in the real time, for example, using total systems that record movements at short intervals automatically or vibrating wire strain gauges with data transferred to the responsible person, as discussed in Chapter 4. Alternatively, periodic examinations using inclinometers, radar or photogrammetry can all be effective.

6.6.1.3 Deep-seated failure

Very large rock slope failures often involve some zones where sliding on discontinuities is happening whilst elsewhere the rock mass may be acting as an isotropic fractured mass in a Hoek-Brown way and in other areas intact rock may be failing. Explaining such complex failures is a much easier task than prediction. Many large failures have been studied in detail, and these cases are probably the best place to look for ideas and inspiration when dealing with large slopes (e.g. Bisci *et al.*, 1996; Eberhardt *et al.*, 2004).

6.6.2 Soil slopes

For soil slopes, where the ground mass can be regarded as essentially isotropic within each unit or layer (stratum), analysis involves searching through the slope geometry, looking for the potential slip planes with the lowest FoS. This can be done easily using available software such as SLOPE-W and Slide. In weathered rock and indeed many soils, stability might well be controlled by adverse relict joints and other weak discontinuities so that a variety of possible failure modes need to be addressed. An example of hazard models requiring particular analysis within cut slopes in Eocene mudstone at Po Chang in Korea is given in Box 6-5.

Box 6-5 Hazard models for a slope, Po Chang, Korea

The slope shown in Figure B6-5.1 is within a development site near Po Chang, South Korea. The rock comprises weak to strong bedded mudstone containing strong rounded concretions. The slope was excavated several years prior to the photograph and in some areas is deteriorating very rapidly, with large screes of disintegrated mudstone debris.

Apart from bedding, the main discontinuities are orthogonal vertical sets of joints, probably formed during burial. There are also conjugate shear fractures, inclined at steep angles. As Figure B6-5.2 shows, the same rock is exposed in unprotected slopes adjacent to main roads on the outskirts of Po Chang. There are evident recent failure scars in some of the slopes.

This case provides an example of how a single slope or series of slopes may contribute several different hazards, each of which needs to be considered in a different way, as illustrated in Figure B6-5.3.

Figure B6-5.1 View of large cut slope in Eocene mudstone near Po Chang Korea.

Figure B6-5.2 Road side cutting though same sequence of Eocene mudstone on outskirts of Po Chang.

Shallow hazards include boulder fall from the concretions, undermined from the continuing ravelling deterioration of the mudstone. Trees may collapse in a similar way.

There is a risk of structurally controlled failure on the steeply inclined conjugate shear set of joints, with vertical joints providing the release surfaces. There was evidence of such failures in some exposures. Finally, there is a risk of large-scale landslide in these steep slopes, involving a generalised slip surface through the closely fractured rock. The question is how to determine an appropriate set of strength parameters for analysis. It might be reasonable to use the GSI approach (Marinos &

Hoek,2000), but the first step would be to collect more empirical data on the way that the Po Chang mudstone behaves regionally and to identify whether there are any large-scale failures that might be back-analysed.

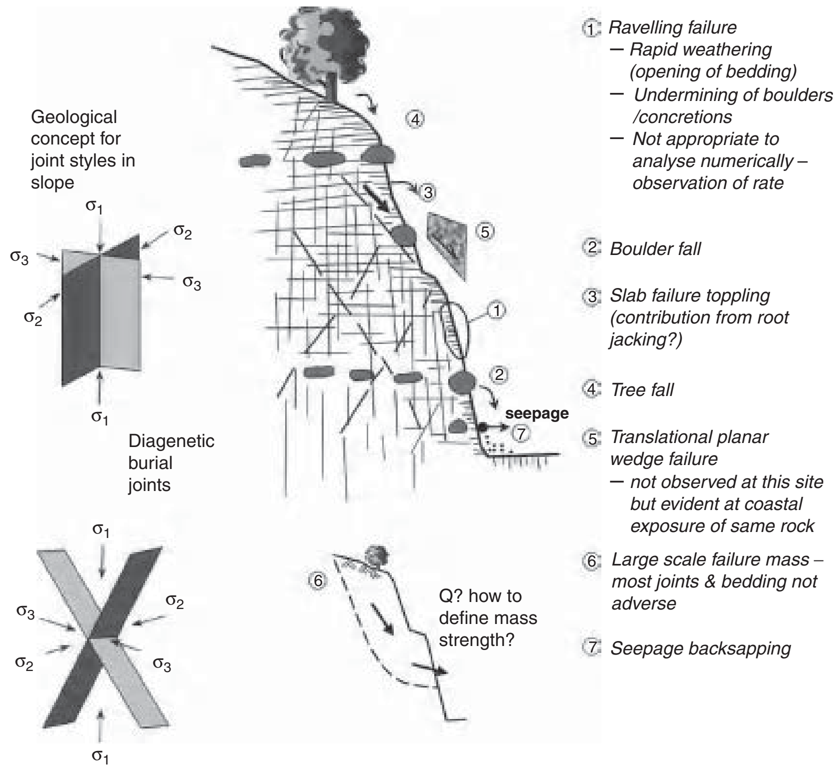


Figure B6-5.3 Ground model for slope. Main discontinuities include bedding which is almost horizontal, orthogonal near-vertical joints and steeply inclined shear joints. Potential failure mechanisms (1 to 7) are identified, each of which need to be considered and assessed individually, both through observation in the field and by numerical analysis where possible.

The simplest type of analysis is undrained in which it is assumed that the soil has a uniform strength, independent of stress level, expressed as cohesion along the potential slip plane. The logic is that any change in normal stress is matched by a change in water pressure so that change in effective stress and frictional resistance is zero. This type of analysis is only appropriate for earthworks in clay, immediately after cutting, and is not considered further here (see also Box 6-3 re pile design).

More generally an effective stress analysis is used where the strength of the soil (or closely fractured rock) is considered to be derived from two components – friction and cohesion, as per the Mohr-Coulomb expression:

$$\tau = (\sigma - u)\tan\phi + c$$

where τ is shear strength; σ is total stress (generally due to weight) normal to the failure plane; u is water pressure reducing σ to an effective stress, σ′; ϕ is angle of friction; and c is cohesion.

Frictional resistance changes with stress conditions, which vary throughout the slope, and to deal with this, a method of slices is used typically to calculate stability. Figure 6.24 shows a slope with the potential failing mass split into four vertical slices. In this diagram, the weights of slices 1 and 3 have been resolved into destabilising shear force, S, parallel to the tangent to the section of slip surface below each slice and a normal force acting normal to the shear surface (N). It is evident that the ratio of S to N varies considerably from one slice to the next. Slices 1 and 2 are being prevented from failing by Slices 3 and 4. The FoS for the slope as a whole is the ratio of the summation of shear resistances beneath each slice to the summation of the shear components. There are many different versions of the method of slices. For some, circular slip planes are assumed, in others, irregular slip surfaces can be analysed (e.g. Morgernstern & Price, 1965). Slice boundaries are generally taken to be vertical and assumptions need to be made regarding the forces at the vertical interfaces between each slice. The method of Sarma (1975) allows non-vertical slices, which gives some flexibility in dealing with more complex geology. Software packages (limit equilibrium) give a range of options regarding the method of analysis and give almost instant answers so the results from the various analytical models can be compared. Sometimes this is done in a probabilistic manner, varying the various strength parameters through their anticipated ranges and distributions (Priest & Brown, 1983). Generally, these analyses are carried out to try to establish that the FoS exceeds some chosen value – typically between 1.2 for a slope with low consequence of failure and 1.4 for a higher risk slope and, empirically, most slopes analysed with such FoS

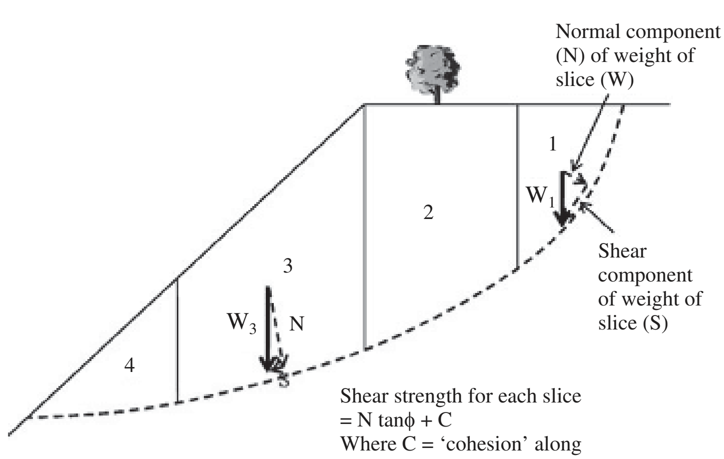


Figure 6.24 The method of slices for slope stability analysis.

will stand safely provided that the ground model is correct – probably in part because of inherent conservatism in most assessments of mass strength parameters (see discussion on disturbance in Chapter 4). More sophisticated analyses can be carried out using generalised representations of soils and their properties in both two and three-dimensions (e.g. FLAC SLOPE and FLAC3D – Itasca), and these software packages allow the engineer to see how the failure develops in a time-stepping manner, which is very helpful. The best use of stability analyses is to test the significance of the various assumptions to the outcome. Lumb (1976) addressed some of the problems of the Factor of Safety approach and advocated that engineers think instead in terms of probability: 'forcing the designer to consider the reliability of all his data and to face up to the consequences of his being wrong'. If, for example, water level is shown as critical to stability, then that should lead to a careful assessment of the need to prevent infiltration and to install drainage systems.

In the partial factor approach of Eurocode 7, each part of the analysis – forces and strength parameters – are factored in a prescriptive manner. Commonly used software packages can cope with this. This approach might be regarded as rather limiting and perhaps giving an incorrect impression that everything is understood and that all factors are always the same. For example, the Eurocode partial factor for cohesion is the same as for friction (1.25), whereas it is common experience that friction can generally be measured or estimated with far more confidence than cohesion and changing assumptions on cohesion can have a disproportionate influence on calculated FoS.

All analyses are of course only as valid as the input parameters and especially the geological and hydrogeological models; if the model is wrong, so will be the analysis. In a study of the failures of several engineer-designed slopes, Hencher (1983e) concluded:

> 'Six of the eight cut slopes that failed had been investigated by drilling in recent years. In five of these cases, important aspects that controlled the failure were missed. In only one case were the true geological conditions recognised, but even then the groundwater levels were underestimated considerably. In all cases, where piezometric data were available and the groundwater level was known by other means, albeit approximately (e.g. observed seepage), the piezometric data did not reflect peak water pressure at the failure surface. This was principally due to failure to observe rapid transient rises and falls in water levels. A further problem was that many of the piezometers were installed at levels where they could not detect the critical perched water tables which developed.'

More recently, Lee & Hencher (2009) document a case study where a slope was subject to numerous ground investigations and analy ses (often in response to some relatively minor failure) over many years, before the slope finally collapsed in a disastrous manner. There were fundamental misconceptions about the geological conditions by all of the investigators. The potential for self-delusion that such methods of analysis truly represent actual stability conditions is expressed a little cynically in the song 'Slopey, Slopey, Slopey' in Box 6-6. Lerouiel & Tavernas (1981) used various classic examples of slope failures and their analysis to demonstrate how different assumptions can lead to different results and explanations.

6.6.3 Risk assessment

A decision needs to be made on whether the risk from slope failure is acceptable or not and whether the cost of engineering works can be justified. A modern approach to assessing the need for preventive measures is to use quantified risk assessment, as described by Pine & Roberds (2005). The project described involved remediation and stabilisation of several sections of high cut and natural slopes dominated by potential sheeting joint failures and by the potential for failure of rock blocks and boulders bouncing down exposed sheeting joints to impact the road below. Design of slope cut-backs and stabilisation measures was based on a combination of reliability criteria and conventional FoS design targets aimed at achieving an ALARP (as low as reasonably practicable) risk target, which, in actuarial terms, translated to less than 0.01 fatalities per year per 500m section of the slopes under remediation. Further examples of quantitative risk calculation are given by Fell *et al.* (2005).

6.6.4 General considerations

Remediation of stability hazards on slopes is often not trivial, especially where the works are to be conducted close to existing infrastructure and implementation of the works can itself increase the risk levels, albeit temporarily. Factors that will influence the decision on which measures to implement include the specific nature of the hazards, topographic and access constraints, locations of the facilities at risk, cost and timing. The risks associated with carrying out works next to active roads, both to road users and to construction workers themselves, need to be addressed (GEO, 2000a). Pre-contract stabilisation works might be needed to allow site access and preparation. Preventive measures such as rock bolting may be carried out at an early stage to assist in the safe working of the site and designed to form part of the permanent works. Options for the use of temporary protective barriers and catch nets to minimise disruption to traffic during the works also need to be addressed, as do contractual controls and alternatives for supervision of the works. Traffic controls may be

Box 6-6 Slopey, Slopey, Slopey (1982)

Figure B6-6.1 Chung Hom Kok, Hong Kong.

– sung (and danced if you wish) to the tune of the Hokey-Cokey.

You put your phi value in
You take your c' value out
You add a bit of suction
And you shake it all about
You do the old Janbu[2] and you turn around
That's what it's all about.

Chorus:
Oh slopey, slopey, slopey
Oh slopey, slopey, slopey
Oh slopey, slopey, slopey
It's so easy
One – point – four[3]!

Written and sung by the GCO Cabaret Stars, 1982

[2] Janbu is the author of a commonly used limit equilibrium method of slices for calculating Factors of Safety of slopes and can be applied to non-circular surfaces. There are two forms: a routine method and a rigorous method (Janbu, 1973). Lumsdaine & Tang (1982) carried out an exercise comparing results of calculations by six Government Offices and 36 others and found a very high proportion of analytical errors and lack of documentation, which of course is over and above any uncertainty in ground model, parameters adopted and assumed groundwater conditions – either positive pore pressure or suction.

[3] A Factor of Safety of 1.4 is generally regarded as an acceptable number to guard against failure in a high-risk slope in Hong Kong (GCO, 1979; 1984).

needed, and in some circumstances it will be necessary to close roads or evacuate areas temporarily, especially where blasting is to be used. The use of a risk register, as piloted for tunnels (Brown, 1999), with clear identification of particular risks and responsible parties, helps to ensure that all hazards and consequences are adequately dealt with during construction. Decision analysis is now widely applied at an early stage to assess whether to mitigate slope hazards (e.g. by rockfall catch nets) or to remediate/resolve the problem by excavation and/or support approaches. If construction of intrusive engineering measures to stabilise hazards might be unduly risky, then passive protection can be adopted instead. A hybrid solution is often the most pragmatic approach for extensive, difficult slopes where some sections might be stabilised by anchors and buttresses, with other sections protected by nets and barriers (Carter *et al*., 2002; Pine & Roberds, 2005).

6.6.5 Engineering options

Some of the options for improving the stability of slopes are illustrated in Figure 6.25 and listed more comprehensively in Hencher *et al*. (2011). These can be split into *passive* options that either deal with the possible failure by controlling surface deterioration at source, or installing preventative reinforcement to increase local factors of safety, or adding walls or buttresses to restrain detached debris before it causes injury or damage, and *active* measures that enhance overall Factors of Safety of larger sections of slope by major engineering works, including cut backs or buttresses or heavy tie-back cable anchors.

6.6.5.1 Surface treatment

Many risks can be mitigated cost-effectively through surface treatment to stabilise or remove relatively small blocks of rock. Surface drainage is important, using adequately sized concrete channels with a fall across the slope and channels down the face that may be stepped to reduce velocity of flow. Further guidance is given in GCO (1984a) and in Ho *et al*. (2003).

There is a temptation to use hard slope treatments such as shotcrete to constrain loose blocks at the slope surface but such measures, if not properly designed, can restrict drainage from the slope, hide the geological situation from future investigators and can themselves cause a hazard as the shotcrete deteriorates, allowing large slabs of shotcrete to detach. Furthermore, shotcrete is increasingly an unacceptable solution for aesthetic reasons and there is a push towards landscaping high visual slopes where safety is not compromised (GEO, 2000b).

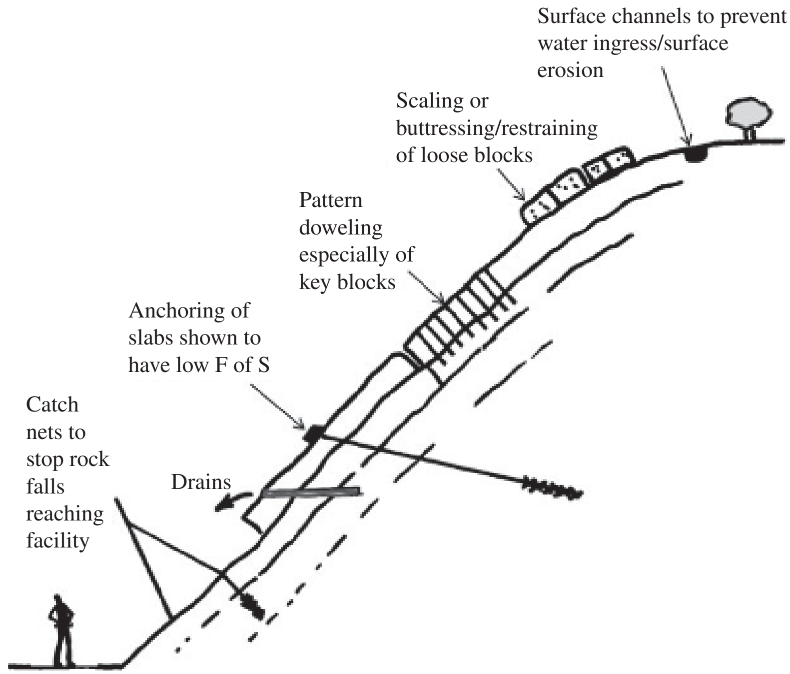


Figure 6.25 Schematic representation of various measures for stabilising rock slopes or protecting public.

Bioengineering is used to generally improve the stability and reduce erosion from natural slopes. Roots bind the soil and vegetation can increase surface runoff. Most bioengineering solutions cannot, however, be relied upon to improve the long-term stability in risky slopes, because vegetation can rot and die or be destroyed by fire. Furthermore, root growth can lead to rock blocks becoming loosened and detached.

6.6.5.2 Rock and boulder falls

Where individual rockfall sources are identified, these can be scaled off, reinforced by dowels, bolts, cables or dentition buttresses and/or netted where the rock is in a closely jointed state. Removing large blocks can be difficult because of the inherent risks associated with breakage techniques, including blasting and chemical splitting, which can dislodge blocks unexpectedly. Care must be taken to protect the public and workers during such operations. The most difficult zones to deal with are those with poor access. Implementing passive or active protection needs to start from safe ground and move progressively into the areas of more hazardous stability.

Rockfall trajectory analysis, using widely available software, allows prediction of energy requirements and likely bounce heights and run-out damage zone extent. Where energy considerations allow, toe zone protection measures, catch benches, catch ditches and toe fences

Figure 6.26 Retaining structures and catch nets to stop natural terrain landslides impacting new road, Lantau Island, Hong Kong.

provide the earliest viable mitigation approach, without requiring access to the slope.

Catch nets or fences can be positioned on-slope or in the toe zone of the slope, depending on energy requirements and site restrictions. An example is shown in Figure 6.26. Where energies computed from rockfall analyses are too high for toe zone protection alone to maintain risk levels below prescribed criteria for highway or rail users, on-slope energy protection fences become a necessity to reduce total energy impact at road level. Where the road (or railway) passes under areas prone to continuous rockfall, an avalanche shelter is commonly used (Figure 6.27).

6.6.5.3 Mesh

Wire mesh is commonly used to restrict ravelling-type rock failure and can be fixed at many anchorage points or can simply hang down the face, fixed with anchors at the top and weighted with scaffold bars or similar at the toe. Mesh (varying from chain-link, triple twist, hex-mesh to ring-net, in increasing order of energy capacity) can be placed by a variety of techniques, ranging from climber-controlled unrolling of the mesh to the use of helicopters.

6.6.5.4 Drainage

Deep drainage can be very effective in preventing the development of adverse water pressures, and this is often a combination of surface protection and channelling of water away from the slope and inclined drains drilled into the slope. Regular patterns of long horizontal drain

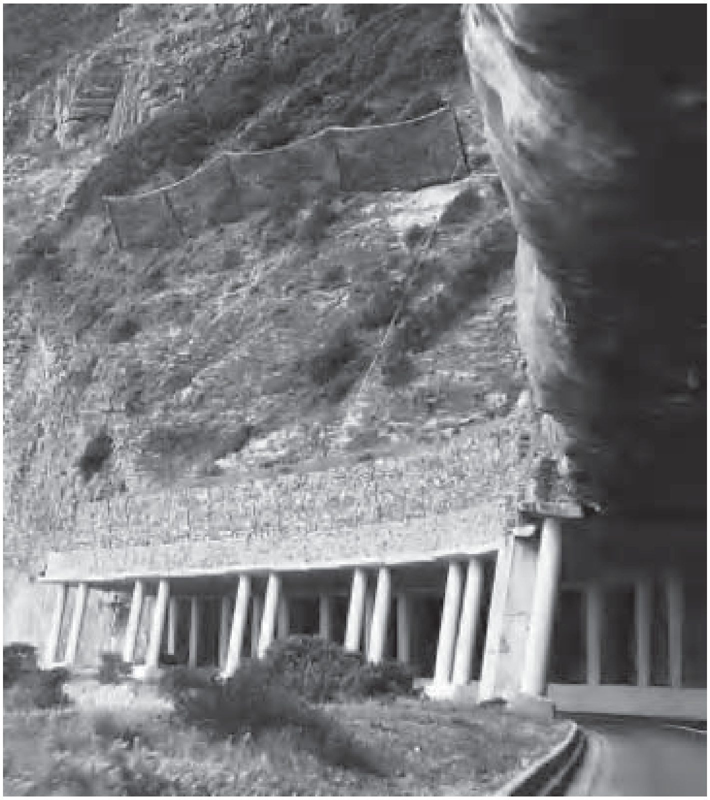


Figure 6.27 Rock-fall nets and avalanche shelter, near Cape Town, South Africa.

holes can be very effective, but all drains will seldom yield water flows, and the effectiveness of individual drains will probably change with time as sub-surface flow paths migrate. Typically, drains comprise plastic tubes with slotted crests and solid inverts, inserted into pre-drilled holes of tens or even hundreds of metres. Inner geotextile liners might be used that can be withdrawn and replaced if they get clogged up. Drains might need to be flushed out periodically. Attention should be made to detailing the drain outlets properly otherwise the slope face may backsap. If not maintained, vegetation can block outlets reducing their effectiveness.

In rock slopes, there is a need to target sub-surface flow channels, many of which will be shallow and ephemeral. The paths may be tortuous and hard to identify and drainage measures can therefore be rather hit or miss. If the exposed joint is badly weathered, the weak material may backsap and possibly pipe, leading to destabilisation, partially caused by lack of free drainage, and careful detailing will be required to prevent deterioration. No-fines concrete, whilst appearing to be suitable to protect weathered zones, often ends up with lower permeability than designed and should not be relied upon without some additional drainage measures.

As an alternative to deep drains drilled into the slope from the surface, drainage adits and tunnels are sometimes used to lower the water table, generally with drainage holes drilled radially into the rock mass from the tunnel walls. Other solutions include deep caissons constructed at the rear of the slope to intercept through-flow, with inclined drains leading away from the slope at their base (McNicholl *et al.*, 1986). Pumped wells are also occasionally used, pumps being activated when water levels reach critical heights within the slope.

6.6.5.5 Reinforcement

Stability can be improved by a variety of reinforcement options. For rough matching joints, provided there has not been previous movement, the interlocking nature provides considerable shear strength. If the joint can be prevented from movement by reinforcing at strategic locations, then full advantage can be taken of the natural shear strength. Depending on configuration, rock may be stabilised by passive dowels, tensioned bolts or cable anchors. Passive dowels allow both mobilisation of a normal force (due to the resistance provided by the fully grouted dowel) plus active shear restraint provided by the steel of the dowels resisting block slide mobilisation (Spang & Egger, 1990).

The Geotechnical Engineering Office in Hong Kong has published some guidelines on prescriptive measures for rock slopes and in particular gives guidance on rock dowelling for rock blocks with volume less than $5\,m^3$ (Yu *et al.*, 2005). In essence, it is advised to use pattern dowels with one dowel per m^3 of rock to be supported, with minimum and maximum lengths of 3 and 6m respectively, and where the potential sliding plane dips at less than 60 degrees. The dowels are to be installed at right angles to the potential sliding plane, with the key intention to allow the dowels to act in shear, whilst also enhancing the normal restraint due to asperity ride during sliding. In practice, dowels frequently need to be used in more variable orientations. Designs must be checked in the field during installation, to check that the perceived ground model is correct. If not, then the design must be revised.

Sub-horizontal cable anchors can be used if capacities larger than about 20 tonnes per reinforcement member are required. Great care needs to be taken to ensure that such tensioned anchors are adequately protected against corrosion, and regular checking and maintenance will be required. Several cases of anchors that have failed due to corrosion are discussed in Chapter 7. For weaker rock and soil, pattern soil nailing is now commonly used. The nails, which typically comprise 50mm or so diameter steel bars connected, as necessary, by couplers every 6m, are usually installed in pre-drilled holes, held centrally by lantern spacers and then pressure grouted over their full length using tubes installed with the nail. Soil nails are usually installed as a passive reinforcement that would only take on load if the slope began to deform prior to failure.

6.6.5.6 *Retaining walls and barriers*

Retaining walls are commonly used to support steep slopes, especially where the slope comprises weak and broken rock and where space is constrained. There are many different types, as illustrated in Figure 6.28.

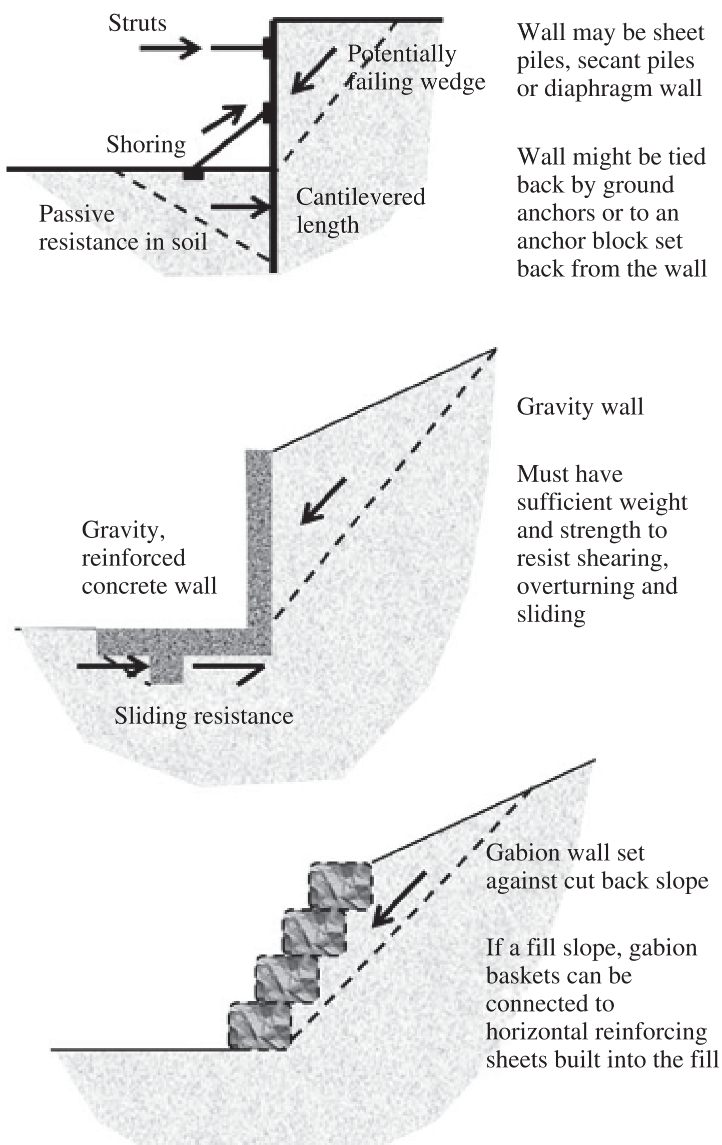


Figure 6.28
Different types of retaining wall.

Figure 6.29 Concrete retaining wall under construction, Hong Kong.

For temporary works, corrugated steel sheets are generally driven, vibrated or pushed into soil prior to deep excavation, with each sheet linking to its neighbour. As the excavation proceeds, sheets are usually braced by a system of struts and waling beams, although they may also rely on depth of embedment. Diaphragm walls formed by concreting deep trenches excavated under bentonite mud are also used as part of temporary works and then may be incorporated in the permanent structure. Permanent retaining structures are often created using piles. Alternatively, where the ground can be anticipated to stand steeply, temporarily during construction, the full slope is cut back and then a wall of concrete constructed at some short distance in front. The space between the wall and the natural ground is backfilled with granular free-draining material, often with geotextile material at the interface, feeding water down to a drain (Figure 6.29). Drainage is very important if the retaining wall is not going to act as a dam. Gabionstructures are made from galvanised steel or, rarely, plastic baskets, backfilled with rock. The main advantages are that they are free-draining, can be landscaped, and they can be made cheaply on site using locally derived rock to fill locally woven baskets. They are therefore very suitable for forming retaining structures or barriers in remote locations (Fookes *et al*., 1985). Deflection structures and barriers are commonly used to divert or retain channelised debris flows away from buildings or roads.

6.6.5.7 *Maintenance*

Whatever the engineering solutions adopted, slopes should be examined periodically for signs of distress and for maintenance such as cleaning out of drainage channels. The requirement for inspection, testing and possible remediation works, should be built into the design of any new slope, with careful consideration for how this is to be achieved. In Hong Kong, the current practice is to prepare maintenance manuals for slopes and to carry out routine engineer inspections at regular intervals. New and newly upgraded slopes are generally constructed with access ladders and often with hand rails provided along berms to allow safe inspection.

6.7 Site formation, excavation and dredging

6.7.1 Excavatability

Site excavation is usually carried out by heavy machinery, and the main questions for the engineering geologist are what machinery would be suitable and whether the rock would need to be blasted first.

Where blasting is restricted, then the contractor might need to use some kind of chemical or hydraulic rock splitter, but the noise levels of drilling and rock breaking might still be a problem. Generally, the factors that will control whether or not blasting is needed are intact rock strength and the spacing between joints (MacGregor *et al*., 1994; Pettifer & Fookes, 1994). As emphasised elsewhere, care must be taken to differentiate between mechanical fractures with low tensile strength and incipient fractures with high strength as this will strongly affect the ability of machines to rip the rock.

6.7.2 Dredging

Dredging (underwater excavation) is commonly carried out for port works, to improve navigation on rivers and as part of other land reclamation projects in providing fill material (Bray *et al*., 1997). There are several types of dredger and these vary in their capacity to deal with soil and weak rock. Where there are few natural fractures in the weak rock, excavations can be difficult, even for the strongest suction-cutter dredgers and then some pre-treatment, normally blasting, will be required. Reviews on dredging practice in various countries, including the USA, UK, Hong Kong and Singapore, are given in Eisma (2006).

6.8 Ground improvement

6.8.1 Introduction

At many sites, the ground conditions are too weak or wet to allow construction by the preferred method or even to allow access by heavy construction equipment. Ground improvement might therefore be carried out, often as an alternative to some engineering solution such as piling, and the engineering geologist should be aware of the techniques that might be employed to deal with a particular site condition (Charles, 2002). Ground improvement might be used in temporary works, such as freezing the ground to allow tunnelling through saturated and potentially flowing materials, or the construction of barriers to water flow, or to restrict vibrations during construction. In other situations, ground improvement might provide a permanent solution such as densification or using chemical additives to provide additional strength.

6.8.2 Dynamic compaction

One of the simplest methods is dynamic compaction, which involves dropping a large weight, up to about 30 tonnes, from a crane, over a regular pattern and then backfilling the depressions with granular material. Further drops are carried out at closer spacing. The depth of improvement depends upon the weight dropped, size of pounder and the height. Typically, a weight of about 15 tonnes dropped 20 m might be expected to improve ground to about 10 m deep (e.g. Bo *et al.*, 2009). The method is most suitable for improving fills and granular soils generally, but sites underlain by clay have also been improved, although consideration must be given to the pore pressures that might be generated and how these dissipate. Generally, the improvement is measured by tests before and after improvement, using techniques such as the SPT, CPT or the Menard pressuremeter that was developed specifically for this purpose (Menard & Broise, 1975). The technique has been applied successfully for quite prestigious projects involving large-scale reclamation, such as Nice Airport. In Hong Kong, it has been used to densify the upper few metres in old fill slopes in an attempt to improve their stability.

6.8.3 Static preloading

If time allows, then an effective way to improve the consolidation characteristics at a site is to preload it, often by placing an embankment of fill material that can be removed again later or re-graded at site, compacted properly in thin layers. The process of consolidation is generally accelerated by introducing a series of vertical drains to increase the mass permeability and allow excess pore pressures to dissipate, monitored using piezometers. The drains can be sand wicks, which are sausages of geotextiles, filled with sand and installed in pre-drilled holes. Other systems include wick drains that are geotextile-covered plastic elements pushed into the ground using a purpose-built machine. At some sites consolidation and strengthening is achieved by a technique termed vacuum preloading; references are given in Charles (2002).

6.8.4 Stone columns

Stone columns can be used to enhance drainage and are installed to depths of 10m and sometimes more. These are formed by using a vibrating poker, pushed into the soil to form a void and then filling the void with gravel and sand, which is compacted in stages using the same vibrating tool (McCabe *et al.*, 2009). Stone columns have been used to increase mass permeability and prevent liquefaction of loose silty sand during an earthquake, although in such a usage settlement will still occur but in a relatively uniform and non-catastrophic manner. Stone columns are also used generally to improve the bearing conditions at a site, the improvement depending upon the ratio of cross-sectional area of stone columns to untreated ground. Groups and lines of stone columns can be used as weak piles to provide support to structures such as oil tanks.

6.8.5 Soil mixing and jet-grouted columns

Clay soils especially, can be improved by mixing with lime slag and cement, either at the ground surface (to prevent erosion in slopes, for example) or in columns or trenches, using hollow-stem augers and similar equipment. The works will improve the bearing capacity of the ground, although the improvement might be difficult to quantify. Stronger columns can be formed by using jet-grouted columns formed using high-pressure grout jets as a drilling string is rotated and lifted from depth. The resulting column of mixed soil and grout can be used to carry structures or to form cut-off barriers to restrict water flow, for example, beneath dams. Jet grouting is sometimes used to form structural members during temporary works construction of deep excavations (Puller, 2003; also see case study of Nicoll Highway collapse in Chapter 7).

6.8.6 Drainage

For deep excavations and tunnelling, it is commonly necessary to lower the groundwater during construction, although there are many factors that must be considered, not least associated settlement of the ground due to increased effective stress and self-weight compaction and consolidation and drying up of land in adjacent properties (Preene & Brassington, 2003). New and steep flow paths through the soil can lead to seepage piping and liquefaction in the floor of excavations. The cheapest and simplest way to lower water is just to let it happen naturally as the excavation proceeds, to channel water inflow to collection sumps and to then pump this water away, although disposal may be an issue on environmental grounds, and pumping from great depth will require a series of pumps at different levels. Active dewatering is generally conducted using well-point systems or submersible pumps in wells. Details are given by Puller (2003). As noted elsewhere, dewatering is often important to the stability of slopes and semi-permanent solutions include drains, drainage caissons and adits. Emergency pumping systems are sometimes set up to be triggered if piezometric levels become dangerously high.

6.8.7 Geotextiles

Geotextiles are fabric or plastic sheets that have many different uses in ground engineering. A few of these are discussed below.

6.8.7.1 Strengthening the ground

To improve site access, sheets of plastic mesh may be laid on the ground and then a layer of gravel placed and compacted on top. The purpose of the geotextile is to prevent the gravel being pushed into and mixing with the underlying soil that may be wet and soft. In this way, temporary road access can be provided. In other circumstances, more complex solutions might be designed involving elements such as stone columns or piles, together with a geotextile grid draped across and linking the structural elements.

Geotextile mats and strips are also used in the design of reinforced earth structures (as are metal grids and strips), as illustrated in Figure 6.21. Basically, the frictional resistance between the soil and grid or mats, placed horizontally and regularly within a fill structure, enhances the overall strength of the soil mass and prevents it failing. Where facing walls are used or the geotextile is wrapped around at the face to prevent soil erosion, the finished structure can be very steep or even vertical.

Plastic grid boxes, infilled with rock cobbles, have been used to form gabion walls as barriers. Care must be taken that the situation is not one where the finished structure can be destroyed by fire and that the deterioration rate is acceptable given the proposed lifetime of the structure.

6.8.7.2 Drainage and barriers

Geotextile sheets are available that are highly permeable but also designed with a mesh size that restricts soil erosion, in the same way as traditional soil filter systems. Geotextiles are therefore used, for example, as part of the drainage system behind concrete retaining walls. Plastic sheets (geomembranes) are used as barriers to water flow, especially for landfill sites. Great care must be taken to ensure that sheets are welded one to the other and that those welds are tested. Membranes must be resistant to and protected from puncturing. Any leakage may be extremely difficult and expensive to rectify at a later stage. In Chapter 7, an example is given where a combination of permeable geotextiles and impermeable geomembranes were used to reduce leachate loss from a quarry used for landfill.

6.8.8 Grouting

Grouting is generally used to increase strength of a rock or soil mass and to reduce permeability (Warner, 2004). It is routinely used below dams to provide a cut-off curtain to restrict seepage through the foundations. A main consideration is the type of grout – usually cement, but sometimes chemical grouts or resin must be used to penetrate low-permeability ground. The pattern of holes to be used, phases of grouting necessary, and pressures to be adopted are also matters for specialist design. Grouting might jack open existing joints in rock or form new fractures in soil and weak rock (claquage). Grouting is sometimes used to correct settlement or other deformations caused by engineering works such as tunnelling (e.g. Harris *et al.*, 1994), but care must be taken that the grouting does not make matters worse, as per the Heathrow Express Tunnel collapse described in Chapter 7.

6.8.9 Cavities

Cavities that engineering geologists and geotechnical engineers need to contend with include natural cavities such as those often found in limestone areas, more rarely in other rock types, including unlikely candidates such as weathered granite (Hencher *et al.*, 2008). The other main problem is mining. Ground investigation for such voids is a matter of careful desk study (including the mining method that might have been used if that is the hazard of concern), focused investigation, possibly using geophysics such as micro-gravity and resistivity and probing, perhaps using percussive drilling to keep the costs down. If and when voids are found, these can be explored and characterised using cameras, echo sounders and radar. In the case of old mine workings, inspection may be required by suitably equipped and experienced persons following proper safety procedures. Depending on their extent, voids may be backfilled, grouted or structurally reinforced, as appropriate. When extensive mine workings were encountered unexpectedly during tunnelling for the high-speed railway from Seoul to Taejon in South Korea, one proposed solution was to construct a concrete structure through the workings, but this was considered politically unacceptable because the public was already aware of the situation. Instead, the route for the railway had to be moved several kilometres at considerable cost and the completed works were abandoned.

6.9 Surface mining and quarrying

Surface mining and quarrying are industries that have strong demands for geotechnical expertise, including engineering geology. Slope design is often very important and the design practices discussed earlier, used in civil engineering, also apply to quarries and open pits and opencast mines. The main difference is that in such enterprises many of the slopes are always changing in geometry as the works progress. One key to success is establishing a safe layout for operations such as crushing and processing plants and for haul roads, whilst avoiding sterilising valuable resources because of the siting of infrastructure such as site offices and treatment plants. Major haul roads also need to be established in a safe manner to avoid disruption to operations if instability occurs. Other faces may well be temporary and are therefore formed at angles that would be unacceptable as permanent slopes in civil engineering. For large open-pit mine operations, the scale of overall slope formation can be huge, extending hundreds of metres, and predicting stability often requires numerical modelling, tied in to monitoring systems. Excavation of rock usually involves blasting and this is a specialist operation as it is for tunnelling. A good review is given in Wyllie & Mah (2004). Key considerations for all blasting operations are fragmentation, to avoid producing large blocks that cannot be handled easily and need secondary breaking operations, avoiding damage to the remaining rock, avoiding overbreak beyond the design profile, safety and risk from flyrock, gases and vibrations.

Waste from mining needs to be disposed of. In open-pit coal mining, the waste rock is backfilled into the void as part of the ongoing operations and nowadays in the UK at least, the final reinstatement of the area is strictly controlled, with every attempt made to simulate the natural countryside as it was pre-operations. Other wastes are often wet and contaminated and held behind tailings dams that should be designed and analysed with just as much care as a civil engineering structure. Unfortunately, this is often not the case, and there have been many major failures worldwide over the last fifty years which have resulted in severe contamination and many deaths (Rico *et al*., 2008).

6.10 Earthquakes

There are four major considerations for design:

1. Local ground failure, e.g. because of liquefaction in loose saturated cohesionless sand and silt.
2. Rupture because of fault movement, which can be significant especially for tunnel design.
3. Ground shaking causing inertial forces. Buildings and slopes are especially at risk from horizontal shaking.
4. Remote hazards. These will include landslides from adjacent land where debris run-out could impact the site, and tsunamis.

6.10.1 Ground motion

Most structures need to be designed to withstand dynamic loading. This includes wind loading (to typhoon levels in countries such as Japan, Korea and Hong Kong), earthquakes and blasting/traffic. The main one of these that requires input from the engineering geologist is earthquake loading. The level of hazard is assessed at the site investigation stage (Chapter 4), and there is often a mandatory design code for a particular country. Alternatively, or as a check, the design team will identify some design earthquake or series of such design events with equal probability of occurrence within the lifetime of the structure. For example, statistical analysis of historical earthquake activity might indicate that there is an equal chance of a magnitude 8 (M8) earthquake at 200 km distance, as a magnitude 5.2 (M5.2) earthquake at 10 km. These earthquakes would probably result in very different ground shaking at the project site. From study of recorded data using

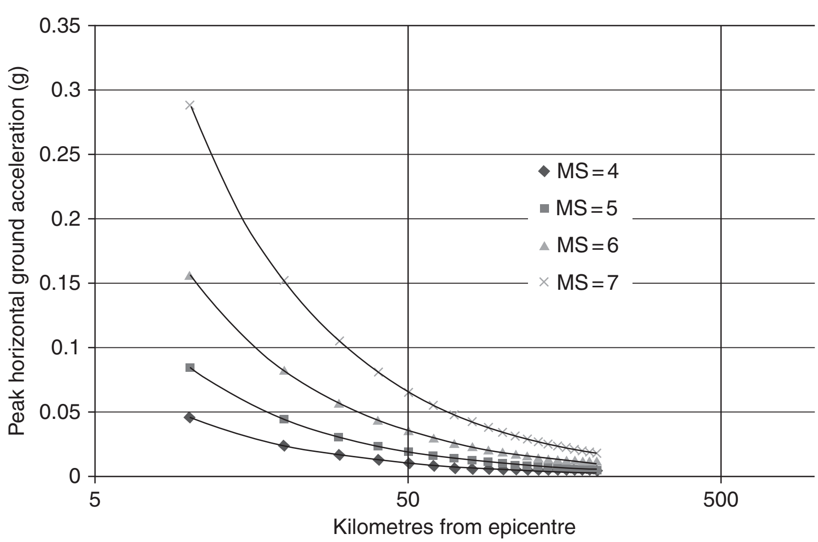


Figure 6.30 Peak acceleration vs. distance for different magnitude earthquakes (European data). Equations given in Ambraseys *et al.* (1996).

strong motion seismographs, attenuation laws have been derived for different parts of the world. Forces are used for engineering design, so acceleration is an important parameter. The equation below has been shown to fit the available European seismic data reasonably well and can be used for prediction (Ambraseys *et al.*, 1996). Data from North America and elsewhere are not very different.

$$\log(a) = -1.48 + 0.266M_s - 0.922\log(r)$$

where a is peak horizontal ground acceleration expressed as a fraction of gravitational acceleration, g, ($9.81\,m/s^2$). M_s is surface wave magnitude and r is essentially the distance between the project site and the earthquake epicentre. Figure 6.30 gives median data and can be refined for degree of confidence and for site characteristics (Ambraseys *et al.*, 1996). Unexpectedly high accelerations do occur, and this is often the result of local ground conditions or topography that amplify the effect, as for the peak accelerations of 1.25g and 1.6g in the abutment of Pacoima Dam, USA, during two separate earthquakes (Bell & Davidson, 1996). The February 2011 earthquake that caused huge damage in Christchurch, New Zealand, involved vertical ground accelerations up to 2.2g and horizontal ground accelerations of up to 1.2g, which are very high for a 6.3M event and can be largely attributed to the very shallow nature of the earthquake (about 5 km, according to the New Zealand Society for Earthquake Engineering).

Peak ground acceleration, although an important starting point, is not enough to give an indication of structural performance. What also matters is the time that the strong shaking continues and the frequency spectrum of the waves carrying the energy. The situation is

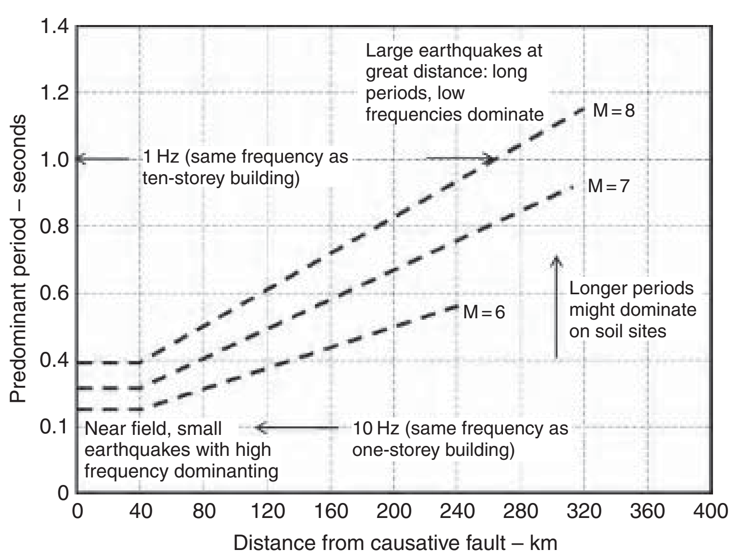


Figure 6.31 Predominant periods in rock for different magnitudes of earthquake at different distances (US data).

complicated by the way that individual structures respond to repetitive dynamic loading, which is a matter of harmonic resonance. Thus, whilst for the M5.2 design earthquake at 10 km the peak acceleration can be predicted from the equation presented earlier as 0.12g and for the M8 design earthquake at 200 km as 0.04g, other characteristics will be very different. Figure 6.31 shows the predominant period in ground acceleration records for western USA (Seed *et al.*, 1968), which indicates that for a near-field M5.2 quake the predominant period might be less than 0.2 secs, whereas for the distant M8 quake the predominant period could be more than 0.8 secs. Furthermore, the duration of shaking will be significantly longer for the large magnitude earthquake (e.g. Bommer & Martinez-Pereira, 1999). The duration of strong shaking for a M5.2 earthquake might be a few seconds. For the Christchurch Feb 2011 M6.3 earthquake, the strong shaking lasted about 12 seconds. For an earthquake of M8, the duration could be over a minute. With longer duration, the potential for amplification will be much greater and fatigue-type failure can occur.

6.10.2 Liquefaction

Liquefaction is a common failure mode in natural soils, fill and sometimes in embankment dams during earthquakes. It occurs in loose saturated cohesionless sand and silt, which, when disturbed, loses its structure and collapses. Because of its low permeability, water cannot escape so natural piping and even general liquefaction occurs as the effective stress and thereby friction reduces to zero. There are many classic examples of whole apartment blocks tilting over and buildings settling. Elsewhere, service pipes float to the surface and sea walls collapse as the retained fill flows into the sea. The potential for liquefaction is readily identified during site investigation. The general rules are:

1. It occurs in un-cemented deposits – fill or geologically recent soil.
2. The most susceptible soils are cohesionless (sands and silts) with a liquid limit less than 35% and water content greater than 0.9 times the liquid limit (Seed & Idriss, 1982).
3. It generally occurs at depths shallower than about 15 m.
4. Generally, SPT N value (corrected) less than 30 (Marcusson *et al.*, 1990) or CPT cone resistance less than 15 MPa (Shibata & Taparasaka, 1988).

Analysis of the hazard might be refined by considering the liquefaction potential vs. the characteristics of a design earthquake, but generally if the area has high seismicity and the granular soil at a site is relatively loose and groundwater table high, then it is probably wise to carry out preventive measures. These might include compaction, grouting or the installation of stone column drains that will help prevent excess pore water pressure development, although they would not prevent settlement. Alternatively, passive mitigation may be the best option – relocate the proposed structures away from the zone of liquefiable soil. If the ground does liquefy, then apart from movement of structures in or on the ground, the settled soil might cause drag down (negative skin friction) on any piles installed through that zone.

6.10.3 Design of buildings

For buildings such as one or two-storey houses, there are certain simple rules that, if adopted, can reduce the risk of failure and would limit injuries, especially in developing nations. These include ensuring that walls are tied together, preferably by reinforced ground beams or beams along the tops of walls (Coburn & Spence, 1992).

For larger engineered structures, these need to be designed to withstand the repeated force waves. As outlined in 6.10.1, given a particular design earthquake one can make estimates about the ground motion characteristics that the structure will have to withstand. These include peak acceleration, predominant frequency and duration of shaking for a given return period earthquake. Typically, the return period used is 1 in 500 to 1,000 years but the choice is rather arbitrary and will depend on the nature and sensitivity of the project and the seismic history. These bedrock ground motions may be modified by the local site geology or topography and estimates of the modified shaking characteristics can be made by dynamic analysis using

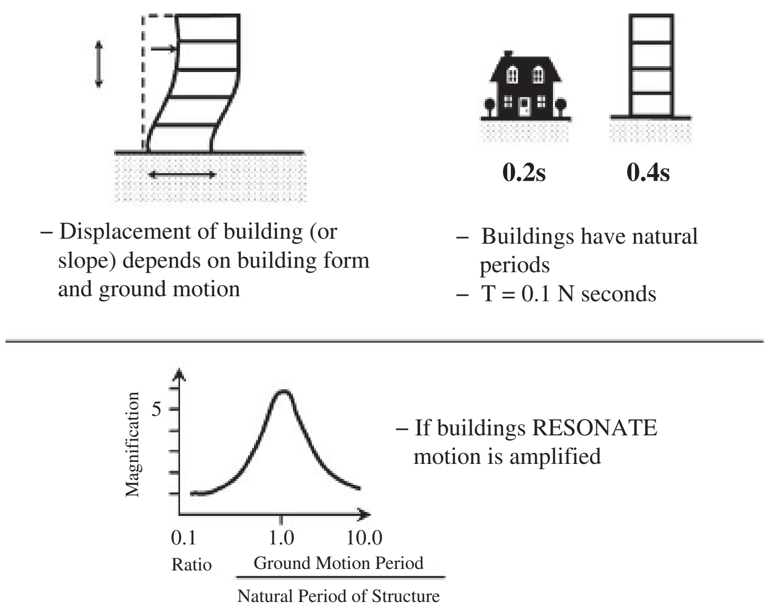


Figure 6.32 Responses of buildings to earthquake shaking.

software such as SHAKE or through reference to published ground motion spectra for particular ground profiles. Generally, thick soft soil profiles may lead to relative amplification of longer period waves. The design ground motion then needs to be applied to the structure. Structures have their own dynamic characteristics and if the incoming frequencies match the natural response frequencies of the structure, then movements may be magnified (Figure 6.32). Structural engineers will take the incoming design earthquake characteristics and calculate the response of the structure. For more frequent, smaller earthquakes, the structural engineer will design the structure as far as possible to behave elastically (no permanent displacement). In the event of an extremely large and less probable event, a structure can be designed to be fail-safe. Redundant elements such as additional steel beams can be included that yield under extreme loads but also change the fundamental frequency of the building, damping the response to the shaking. Other options are to put a building on springs of some kind or to include hydraulic actuators or pendulums that again reduce the structural shaking. An example of an innovative aseismic design is the foundations for the Rion–Antirion Bridge constructed in Greece in 2005. The cable-stayed bridge, with five main spans extending 2.25 km across a fault zone, was designed to withstand horizontal accelerations of 0.5g at ground level and up to 2 m offsets between adjacent towers. Underlying each tower is thick soil and the depth of sea is up to 65 m. The towers were founded on 90 m diameter cellular structures placed on a 3.6 m layer of gravel placed on the natural soil, which was reinforced by up to 200×2 m diameter tubular piles to depths of 30 m. The foundation structure is not attached to the piles; the gravel acts as a fuse, limiting the transfer of load to the superstructure. The piles in the underlying soil are there to prevent rotational bearing failure. Details of the design of the foundations are given by Combault *et al.* (2000) and further references are given at the web page for the bridge.

Two recent earthquakes, however, show that even with good design practice, earthquakes can cause damage to a level that is not anticipated. As a result of the February 2011, Christchurch, NZ, earthquake, many small one-and two-storey buildings were destroyed or badly damaged, as one might expect near the epicentre of an earthquake with magnitude exceeding 6.0, where the ground motion might be expected to be dominated by high frequencies. Widespread liquefaction was also a major contributor to the damage of these smaller buildings. However, for this earthquake, because of its shallow nature and possibly other factors that served to concentrate and amplify the ground motion, unexpectedly large accelerations and forces were generated. In the case of the March 2011 earthquake that struck NE Japan (east of Honshu), most engineered buildings on mainland Japan withstood the very strong shaking associated with this 8.9 or even 9.0M earthquake (10,000 times as strong, in terms of overall energy release, as the Christchurch earthquake) and this is testament to the skill and knowledge of the civil engineer designers. The huge damage and large number of deaths caused by the Japan earthquake resulted from a 10m high tsunami wave that came ashore and destroyed whole villages. Regarding engineered structures, several nuclear power stations had been constructed along the shoreline in the impacted region. The structures apparently performed well in terms of withstanding seismic shaking but severe damage did occur because of failure of cooling systems. The initial shaking caused safe shutdown of the reactors, as is the required procedure for nuclear power stations impacted by a major earthquake, but the loss of electrical power stopped the flow of cooling water required to prevent the fuel rods overheating. Backup diesel generators kicked in and provided the necessary power for an hour or so but then they failed because of the tsunami. In hindsight, no doubt the secondary power sources could and should have been designed to survive inundation, as they are for more modern installations, and the risk properly identified using an event tree approach.

6.10.4 Tunnels

Tunnels and mines tend to be safer than surface structures during earthquakes, and this safety increases with increased depth (Power *et al.*, 1998). Except where the tunnel passes through particularly poor ground or intercepts active faults, earthquake resistant design is generally not a high priority. Of course, where the support in a tunnel is inadequate or marginal under static loading conditions, then earthquake shaking might well trigger failure. This is especially true at portals of tunnels; landslides and especially rockfalls are very commonly triggered by earthquakes, as discussed in the next section. Failures in some tunnels, and especially the failure of Daikai subway station during the Kobe earthquake in 1995, have caused a rethink on seismic stability of underground structures. Hashash *et al.* (2001) provide a very useful review and examples of aseismic design. A reinforced concrete lining should have significantly better seismic resistance characteristics than an unreinforced lining. If the tunnel intersects a fault that is suspected of being active, then special measures will be required or, preferably, the fault avoided. Key considerations are the estimated magnitude of the displacement and the width of the zone over which displacement is distributed. If large displacements are concentrated in a narrow zone, then the design strategy may be to enlarge the tunnel across and beyond the displacement zone. The tunnel is made wide enough such that the fault displacement will not close the tunnel and traffic can be resumed after repairs have been made. In some cases, an enlarged tunnel is constructed outside the main tunnel and the annulus backfilled with weak cellular concrete or similar. The backfill has low yield strength to minimise lateral loads on the inner tunnel liner, but with adequate strength to resist normal ground pressures and minor seismic loads. If fault movements are predicted to be small and/or distributed over a relatively wide zone, it is possible that fault displacement may be accommodated by providing articulation of the tunnel liner using ductile joints. This detail allows the tunnel to distort into an S–shape through the fault zone without rupture, and with repairable damage. This may not be feasible for fault displacements more than 75–100 mm. An alternative approach is to accept that damage will occur and to make contingency plans to control traffic and to carry out repairs as quickly as possible in the event of a damaging earthquake.

6.10.5 Landslides triggered by earthquakes

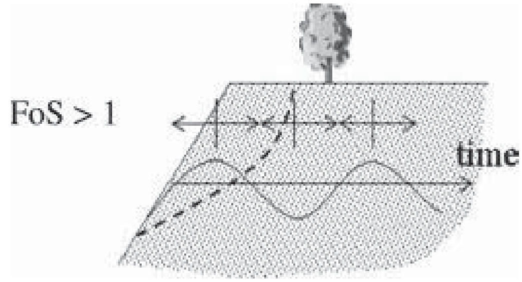
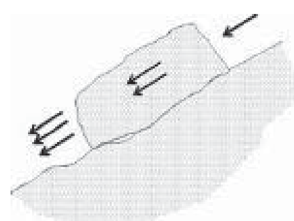
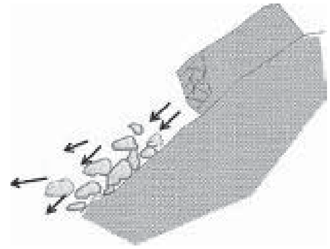
Landslides are commonly triggered by earthquake shaking, especially in mountainous areas. The Wenchuan earthquake in Sichuan Province, China, of 12 May 2008, was very large (M8) and quite shallow (14 km) and the active faults ran through populated valleys surrounded by high slopes. Landslides, including rockfall, caused more than 20,000 deaths, with one individual landslide killing more than 1,600 people (Yin *et al*., 2008). One of the main consequences was the damming of streams, which necessitated emergency engineering works to lower the water levels in the lakes that formed behind the landslide debris before they were overtopped or burst uncontrollably.

6.10.5.1 Landslide mechanisms

Slopes affected by strong earthquake shaking can be categorised in three classes, as set out in Table 6.4. These are:

1. Stable slopes: these are defined as situations where the shaking is not strong enough to cause permanent displacement in a slope. This may be because the peak forces are insufficient to overcome the strength of the ground or because different parts of the same slope are out of phase so that whilst some parts are being driven towards failure, other parts are being accelerated in the opposing direction.
2. Permanently displaced slopes: the key aspect of dynamic loading, whether it is from earthquakes or blasting, is its transient nature. The waves pass through the ground and induce inertial forces. In the same way as discussed in Chapter 5 regarding a laboratory experiment (Figure 5.27), at a critical acceleration (k_c) a slope will start to move. The continued positive acceleration above critical will cause the displacement to increase in velocity. However, after

Table 6.4 Performance of slopes under dynamic loading

Class	Condition	Details and examples
1. Stable	Acceleration less than that necessary to cause permanent displacement	
2. Permanent displacement but stable	FoS post-quake > 1.0 Damage may allow deterioration and later collapse	 Tension cracks resulting from Erzincan earthquake, Turkey, 1992
3. Failure	Metastable condition so that acceleration leads to catastrophic failure directly Reducing shear strength so that FoS is less than 1.0 after earthquake shaking has finished. Deteriorating condition following earthquake Rise in water pressure due to collapse of soil structure or regional changes in hydrogeological conditions	 e.g. Tsao ling reactivated landslide, Chi Chi earthquake, Taiwan, 1999  Rockfalls and other landslides may continue for days after an earthquake

a short time (typically a fraction of a second), the exciting acceleration will decrease and then change direction so that the inertial force is back into the slope. This will stop the movement unless the slope is metastable, as discussed later. Sliding friction can be lower than residual (Hencher, 1977, 1981a; Crawford & Curran, 1982; Tika *et al.*, 1990), and by employing pessimistically low shear strength, total displacement can be calculated for a series of acceleration pulses and this used as part of a design decision. Generally, even for a very large earthquake, the permanent displacement in a slope directly attributable to inertial loading will be small, of the order of millimetres or centimetres (Newmark, 1965; Ambraseys & Srbulov, 1995). Nevertheless, small permanent displacements will make the slope prone to accelerated weathering and deterioration if not protected or repaired.

3. Failed slopes: catastrophic landslides during earthquakes can be the result of four different conditions, viz:

   - *Low residual strength*. The inertial displacement during the earthquake reduces shear strength to a residual value so that even after the earthquake shaking, the slope continues to move. Examples of large-scale failures involving sliding on bedding planes with reducing strength are described for the Chi-Chi earthquake (1999) by Chen *et al.* (2003) and Chigira *et al.* (2003), and for the Niigata earthquake (2004) by Chigira *et al.* (2006).
   - *Deteriorated state*. The structure of rock or soil mass is disturbed so that it collapses and a flow can develop.
   - *Geometrically unstable equilibrium*. The initial displacement caused by the earthquake shaking results in unstable equilibrium. A typical example is rockfall from exposed rock cliffs. Once displaced, the rock will fall, sometimes as a progressive failure several days after the earthquake. Rockfalls may become entrained and develop into debris avalanches.
   - *Water-induced failure*. Firstly, loose saturated soil can collapse and liquefy down to depths of about 15 m on slopes inclined at only a few degrees. The collapsed material can spread and flow. As a second mechanism, the general groundwater flow paths can be affected by earthquake loading and this can trigger slope failures.

6.10.5.2 Empirical relationships

Keefer (1984, 2002) identifies 14 individual types of earthquake-induced landslide. The three main categories are:

1. Disrupted slides and falls: these include highly disrupted landslides that move down slope by falling, bouncing or rolling, or by translational sliding, or by complex mechanisms involving both sliding and fluid-like flow. They typically originate on steep slopes, travel fast and can transport material far beyond the slope in which they originate. Other than large rock avalanches, failures in this category are thin with initial failure depths less than 3 m.
2. Coherent landslides: these include translational slides and rotational slides. Such failures are typically relatively deep seated (greater than 3 m), slow moving and displace material less than 100 m.
3. Lateral spreads and flows: fluid flow is the dominant mechanism and this mode of failure is typical of liquefied soils.

The most common failures, according to Keefer, are rockfalls, rockslides and disrupted soil slides. This follows from the analysis of Table 6.4 where it can be seen that significant landslides will only occur where there are predisposing factors such as a topographic setting that is in unstable equilibrium or strain softening (due to collapsing structure or low residual strength, for example, through the loss of rock bridge-cohesion during the earthquake shaking). Keefer compiled data from many earthquakes and plotted the area affected by earthquakes vs. magnitude of the earthquake. The upper bound is rather well defined. For a magnitude M5, the affected area might be about 100 km^2, 1,000 km^2 for M6, and 10,000 km^2 for M7. Keefer also presents data on the maximum distance of landslides triggered by earthquakes of given magnitude. He provides separate upper bound curves for disrupted, coherent and flow-type failures. Disrupted landslides such as rockfalls, which are the most common type of earthquake-triggered landslides, are also shown as the most likely to occur at far distances from the epicentre. Rodriguez (2001) has carried out a further review of data, including more recent data from Japan, and his data demonstrate the considerable scatter that can be expected and therefore the difficulties in prediction on a site-specific basis. For example, some M7 earthquakes only cause landslides within an epicentral distance of 10 km whereas others of the same magnitude cause landslides 200 km away. This might be attributed in part to resonance effects associated with ground frequency spectra and duration, as for buildings (Hencher & Acar, 1995).

6.10.6 Slope design to resist earthquakes

Traditionally, and in most software packages, there are two main approaches to slope design to withstand dynamic loads (mostly earthquakes). The options for landslide prevention are essentially the same as for the static condition (change geometry, reinforce, reduce water pressure, protect the site below or move the facility at risk).

6.10.6.1 Pseudo-static load analysis

One approach is simply to include a horizontal inertial load into the analysis (some authors argue for an inclined force but it really makes little difference considering the inexact nature of the method) and to determine whether or not the FoS reduces below 1.0. The problem with this approach is that if one includes the peak predicted particle acceleration (say from equation 6.1) then very often the slope will be shown to fail, whereas in reality the permanent displacement would be negligible because of the extremely short time that acceleration would be acting. As confirmation, many vertical slopes in quarries are acted on by accelerations approaching or exceeding 1g during production blasting, but landslides due to blasting are very rare. Engineers therefore often choose to use some arbitrarily reduced acceleration, such as a nominal 0.1g, as a pseudo-static force in the stability analysis to check that the slope (or dam) has some degree of resistance to horizontal loading, but this is clearly rather unsatisfactory.

6.10.6.2 Displacement analysis

As discussed earlier, given a predicted acceleration against time record, it is straightforward to calculate the likely displacement that might be caused in a slope during an earthquake, and there are options to do so in software such as SLOPE/W. Those displacements will always be small, however, no matter how large the earthquake, and what matters more is the residual state of the slope after the earthquake – is there a situation where the ground is strain softening or is it in unstable equilibrium? These are considerations for the engineer, who must decide whether additional reinforcement might be necessary or other protective measures such as nets and barriers. Other software such as FLAC and UDEC (Itasca) can be used to study the seismic susceptibility of slopes. These being time-stepping software, the mode of failure can be identified, expressed visually and perhaps as a movie. It might, for example, be possible to test the potential failure mechanism of soil nails during an earthquake, each nail modelled specifically. That said, as ever, the models can only be as good as the input data and results will only be indicative.

6.11 Construction vibrations

6.11.1 Blasting

Blasting causes noise, ground vibrations, air overpressure and flyrock. All of these can be controlled – generally by using less or different types of explosive and limiting the number of charged drillholes that are detonated at the same time. In particular, using millisecond delays between lines of drillholes will reduce the vibration level considerably. Details are given in Dowding (1985) and many other publications.

Safety is a major issue and an engineering geologist working in a situation where blasting is being conducted may well be involved in blast monitoring, checking fragmentation and reviewing the overall suitability of the blast design, given the changing geological situation as the rock is excavated.

6.11.2 Piling vibrations

The other major source of potentially damaging vibrations in civil engineering is from driven piles. Damaging levels are generally limited to about 10 m distance, although this depends on the sensitivity and state of repair of the structure. Predictions can be made using empirical formulae into which the main inputs are hammer energy and distance (Head & Jardine, 1992), but these are rarely very accurate.

6.12 Numerical modelling for analysis and design

6.12.1 General purpose

There are two main groups of programs commonly used: finite element (FE) and finite difference (FD), time-stepping type software. PLAXIS is a general purpose FE package that allows geotechnical situations – foundations, slopes or tunnels – to be modelled. The model is set up and run to give a quick solution to complex equations – perhaps of deformation or calculation of Factor of Safety of a model that is split into elements – mostly triangular. It can also be used to model fluid flow. As with all sophisticated software, it should only be used by those knowledgeable of the underlying mechanics and the way these are dealt with within the computer program. Following the Nicoll Highway collapse discussed in Chapter 7, it was established that there had been a mistake made in the manner in which the design of the diaphragm walls was carried out using an inappropriate soil model. The same problem would have arisen for any finite element package used in this incorrect manner – it is not unique to PLAXIS. The mistake resulted in excessive deformation of the walls and an under-design of their moment capacity, although these two effects did not have any influence on the final failure. The mistake was in adopting effective stress strength parameters in a Mohr-Coulomb model under undrained conditions and expecting the Mohr-Coulomb model to predict an appropriate undrained strength. For clay, such as the Kallang Formation at the Nicoll Highway site, undrained strength is a function of stress history, in particular overconsolidation ratio, because this determines whether a soil will attempt to contract or dilate as it is sheared in an undrained manner and thus generate positive or negative pore pressures during shearing, which in turn decreases or increases the strength. The Mohr-Coulomb model does not consider dilation or contraction during undrained shearing and, as such, cannot model, for example, soft clays (or dense sands) under undrained conditions. If you use a Mohr-Coulomb model for undrained conditions, then you simply use the undrained strength to control failure conditions and not the effective strength parameters. The Nicoll Highway Committee of Inquiry Report on the collapse includes a well-written section on the problem with the Mohr-Coulomb model (Magnus *et al*., 2005).

The finite difference program FLAC is probably the second most generally used software for geotechnical design.

Until recently, the program was quite daunting, requiring individual commands to be typed in, but recent versions have a graphic interface, which makes things easier. As for PLAXIS and other sophisticated programs, a great deal of knowledge and understanding is needed if reasonable results are to be achieved. For example, the model must first be set up with proper boundary conditions and brought to equilibrium as natural ground before any engineering works such as excavation are simulated. FLAC progressively calculates and checks solutions. Intermediate stages can be calculated, saved and expressed graphically as a movie which can illustrate how strains are developing with time. FLAC, like its sister program UDEC, can cope with large displacements, more so than typical FE analyses. FLAC is used mainly for soil and rock that can be characterised as continua. UDEC is used for fractured rock and each fracture or set of fractures can be specified individually in terms of geometry and engineering parameters. Both UDEC and FLAC can be used for foundation design, tunnels and slopes.

Other commonly used software include the suite produced by Rocscience, such as Phase^2, and their use is discussed in detail in Hoek *et al*. (1995) and at http://www.rocscience.com/education/hoeks_corner.

Many authors, experienced in the development and use of software, have recommended that sophisticated software should be used in an investigatory way, using many simple models to check sensitivity to assumptions rather than trying to prepare a single complex model in an attempt to simulate all aspects of a situation at the same time (Starfield & Cundall, 1988). Swannell & Hencher (1999) discuss the use of software specifically for cavern design.

6.12.2 Problem-specific software

Many suites of software have been developed for particular purposes. SLOPE/W and SLIDE, for example, are commonly used for routine design of slopes. The software calculates stability employing the method of slices, as discussed in section 6.6.2, and gives instant solutions for FoS for a wide range of potential slip surfaces, the broad geometries of which are specified by the operator. Controlling factors such as slope geometry, strength parameters and groundwater conditions can be varied rapidly, allowing sensitivity analysis. Structural elements such as soil nails and rock bolts can be included in the models. There are many similar packages available, all of which are verified and validated against standard mathematical solutions. There is a danger that the ease of use of such software in sensitivity analysis, varying a range of likely parameters, can give a misplaced confidence that all possible conditions have been dealt with. If the ground model is seriously wrong, the results will be meaningless.

Other specialist software packages are used for particular design tasks such as rockfall trajectory analysis, stresses around tunnels, pile design and groundwater and contaminant migration modelling. Details of many of these are reviewed at the web page maintained by Tim Spink: http://www.ggsd.com. Most engineering companies also have in-house spreadsheets (often based on EXCEL) used to solve common analytical problems.

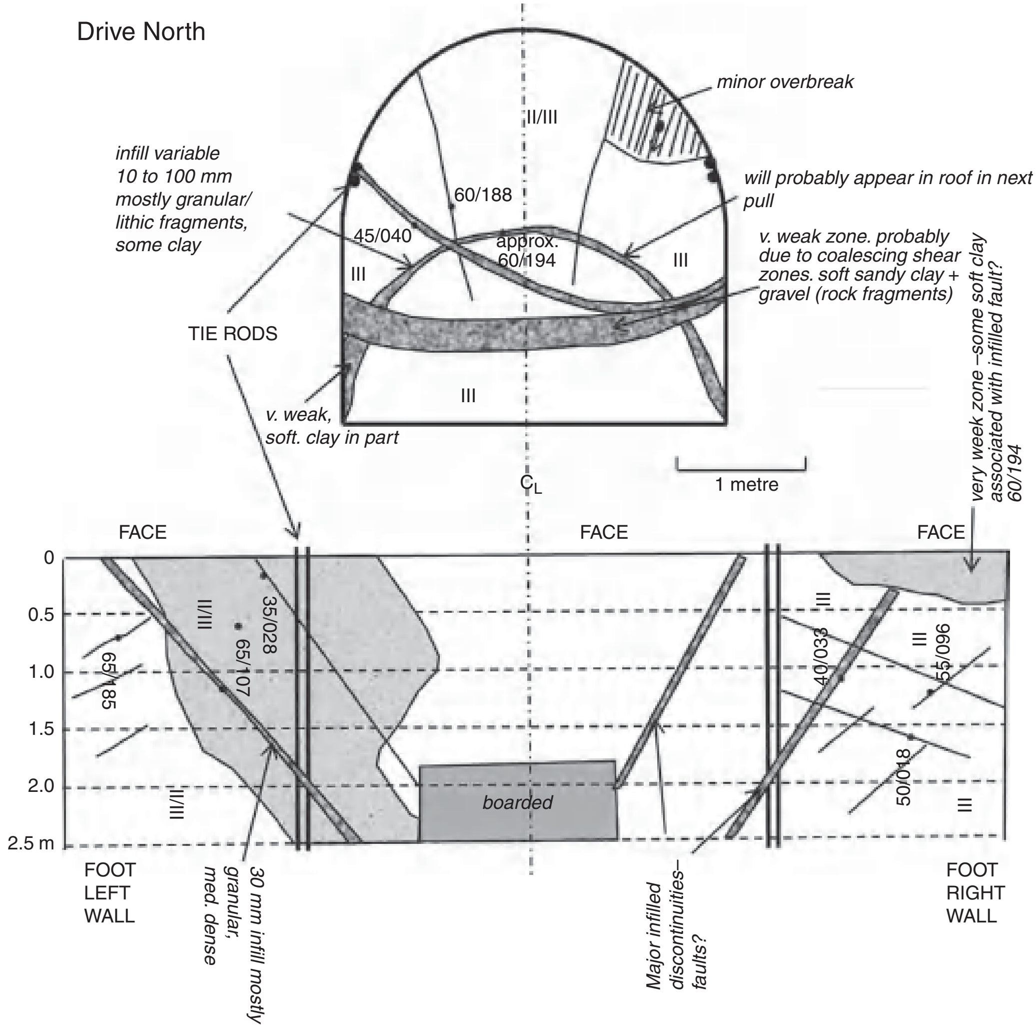
6.13 Role of engineering geologist during construction

6.13.1 Keeping records

Engineering geologists on site should keep careful records as works advance, using daily notebooks. Excavations should be examined, described and photographed as necessary. It is often useful to take photographs and use these as the base for overlays on which to record features such as geological boundaries, strength of materials, discontinuity orientations and style and locations of seepage. Such records will be very helpful in the case of any future disputes over payment or if anything goes wrong. Pairs of photographs taken some distance apart can be used to allow a 3D image to be viewed, and this is particularly useful where access is difficult or hazardous. Where discontinuities are measured, it is important to record the location. In tunnels, description proformas are commonly used as a permanent record, agreed and signed by the contractor and supervisor, as illustrated in Figure 6.33.

6.13.2 Checking ground model and design assumptions

It is fundamentally important that design predictions are checked during construction. Design is usually based on widely spaced boreholes and the interpretation is almost certainly going to be oversimplified. Often this does not have any major consequence but sometimes it does so and the engineering geologist on site should be alert to any indications that the ground model is incorrect or inadequate. Any changed conditions should be flagged up quickly to the designers so that necessary rectifications can be made.



ROCK TYPE	DETAILS	ADDITIONAL NOTES
NAME	*GRANITE*	*v. weak zone in lower part of left wall and across face. soft clay in part. Thick zones up to 0.5 m are at intersects of infilled joints. One dipping south (194), up into face has softened up invert 1.5 m back from face. Possibly will cause overbreak on next pull.*
COLOUR	*Pink, mostly discoloured brown*	
STRENGTH	*moderately strong. locally strong/ mod. weak*	
WEATHERING	*III occ. II*	
JOINT DETAILS	*mostly 300 mm, incipient closer Generally tight*	
DIPS/ AZIMUTHS	*Several major infilled discontinuities/shear zones. also minor clay infilled*	
WATER INFLOW	*Damp, no inflow*	CHAINAGE: 95.94
ADD COMMENTS		AGREED RECORD: 15.9.89
		WH SH

Figure 6.33 Example of agreed record of ground conditions in a tunnel. Queens Valley Reservoir diversion tunnel, Jersey, UK.

In rock slope design and construction, the fundamentally important features of discontinuity orientation, lateral persistence, roughness and infill, can only be surmised from typical ground investigation data. Ground models and assumptions need to be checked as the rock is exposed. There are many cases of rock slope failure where sliding occurred on features that were exposed during construction but were either not mapped by the site staff or the significance not recognised. This can be a very difficult situation in that even where rock is well exposed, the presence or otherwise of rock bridges or steps along adversely oriented discontinuities can only be guessed at. Care should be taken to note how easily the rock is being excavated and how stable or otherwise temporary slopes are, as such information will be helpful in judging the risk.

6.13.3 Fraud

As a final note, the engineering geologist should be aware that fraud does occur in civil engineering, as in other walks of life. Whole boreholes in ground investigations have been known to be fictitious, let alone individual test runs. Cases are known where core from one site is placed into core boxes at another site. Tests are sometimes not carried out as specified or results falsified. Data on foundations are sometimes made up – for example, depths, materials used and test results (Hencher *et al.*, 2005). Engineering geologists need to be aware that such practice does occur, albeit rarely, and remain alert in their supervising duties.