# GEOTECHNICAL ENGINEERING OF DAMS

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# 3

# Geotechnical questions associated with various geological environments

As explained in Chapter 2 and later in Chapter 4, site investigations for a dam need to be undertaken with a good understanding of the local and regional geological environment and the investigations should be aimed at answering all questions known to be of relevance to dam construction and operation in that environment. This chapter discusses twelve common geological environments in which dams have been built and derives check lists of geotechnical questions of specific relevance to each.

It is important that readers appreciate the limitations of these generalisations and check lists. The lists refer simply to features that might be present because they have been found during construction at many other sites in similar environments and because geological reasoning suggests that they could be present. At any particular site the actual geological conditions found will have been developed as a result of many geological processes acting at different times over vast periods of geological time. If some of these processes have been very different from those assumed in the "general" case, then some or even all of the generalisations may not be valid at that particular site.

For further accounts of regional and local geological enviromments, with derived predictions of site conditions, see Fookes (1997) and Fookes et al. (2000).

## 3.1 GRANITIC ROCKS

Included under this heading are granite and other medium or coarse grained igneous rocks. Most rocks of these types have been formed by the cooling and solidification of large masses of viscous magma, generally at depths of greater than 5 km below the ground surface.

### 3.1.1 *Fresh granitic rocks, properties and uses*

In unweathered (fresh) exposures, granitic rocks are usually highly durable, strong to extremely strong (substances) and contain very widely spaced (greater than 2 m) tectonic joints in a roughly rectangular pattern (Figure 3.1). Many of these joints are wholly or partly healed by thin veins of quartz, or quartz/felspar mixtures. Sheet joints are common but, as they are almost parallel to the ground surface, they may be difficult to detect during surface mapping.

Fresh granitic rocks are commonly quarried for rip-rap, rockfill and concrete aggregates, but mica-rich granites may be unsuitable for use as fine aggregates in concrete due to excessive amounts of fine, platy particles in the crushed products.

### 3.1.2 *Weathered granitic rocks, properties, uses and profiles*

Chemical weathering of granitic substances usually causes cracking at the grain boundaries and decomposition of the felspars and ferromagnesian (dark) minerals, leaving quartz grains essentially unaffected.

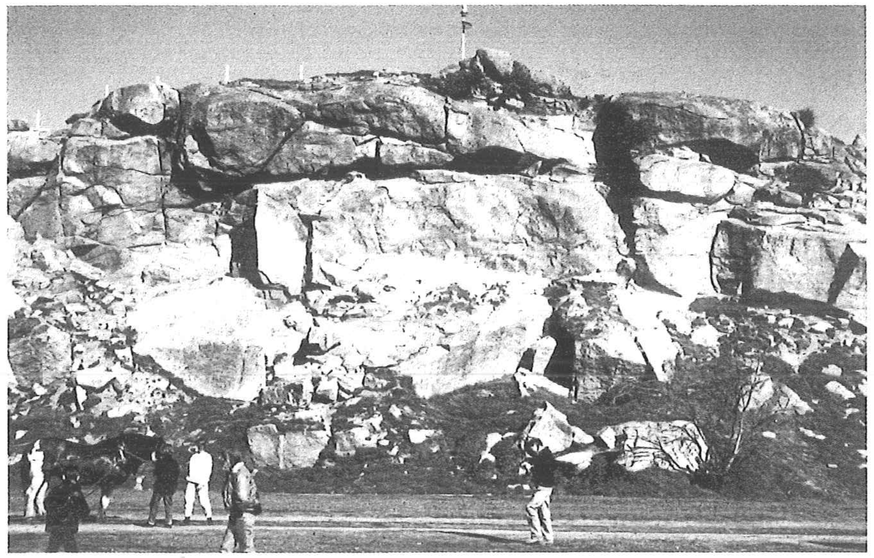


Figure 3.1. Granitic cliff showing 3 sets of joints approximately at right angles to one another.

Table 2.6 is a practical descriptive classification scheme for weathered granitic rocks. When extremely weathered (i.e. soil properties) most granitic materials are silty or clayey fine gravels or sands (GM, GC, SM or SC). *In situ* these materials are usually dense to very dense, but in some tropically weathered areas, where quartz has been partly or wholly removed, they are more clay rich and of low density. Somerford (1991) and Bradbury (1990) describe low density, extremely weathered granitic materials at Harris Dam, Western Australia.

The extremely weathered materials often make good core or earth fill materials and where the parent rock is very coarse grained the resulting gravels can make good quality road sub-base for sealed roads or base course for haul roads.

The silty nature of some extremely weathered granitic rocks often causes them to be highly erodible, when exposed in excavation and when used in fills. At Cardinia earth and rockfill dam near Melbourne extremely weathered granite is dispersive and where exposed in the storage area shoreline has required blanketing with rockfill to prevent erosion and subsequent water turbidity problems.

Lumb (1982) describes engineering properties of granitic rocks in various weathered conditions.

Typical weathered profiles in granitic rock masses are shown on Figures 2.17 to 2.22. The chemical weathering is initiated at and proceeds from the ground surface and from sheet-joints, tectonic joints and faults, causing the roughly rectangular joint-blocks to become smaller, rounded and separated by weathered materials. Thus the profile grades usually from residual granitic soil near the surface to fresh rock at depth, with varying amounts of residual "boulders" of fresh or partly weathered rock occurring at any level. Fresh outcrops or large fresh boulders at the ground surface may or may not be underlain by fresh rock. It is not uncommon to find that weathering has occurred beneath such outcrops, along sheet joints, gently dipping tectonic joints, or within previously altered granitic rock (Figure 2.22). Understanding of this potential for variability in weathered granite profiles is important not only for dam foundations, but also when planning and operating either a quarry for rockfill, rip-rap, filters or aggregate, or a borrow pit for earth fill or core materials.



### 3.1.3 *Stability of slopes in granitic rocks*

Active landsliding, or evidence of past landsliding, is relatively common in steep country underlain by weathered granitic rocks, particularly in areas with high rainfall. Brand (1984) describes the widespread occurrence of natural and man-induced landslides in weathered granitic rocks in Hong Kong.

Although some landslides in weathered granitic rocks no doubt occur by failure through the fabric of extremely weathered material, it is probable that in many landslides failure occurs wholly or partly along relict joints or other defects (see Chapter 2, Section 2.10 and Figure 2.28).

The landsliding at Tooma and Wungong dams (Sections 2.10.3.2 and 2.10.3.3) occurred along localised weathered zones along pre-existing defects and past movements had occurred well into rock masses which were dominantly slightly weathered.

### 3.1.4 *Granitic rocks: check list*

- Concealed sheet joints?
- Fresh rock outcrop; does it extend down into fresh rock?
- Chemically altered zone(s)?
- Fresh granite "boulders" within extremely weathered materials?
- Extremely weathered materials, suitable for impervious core? road pavements? highly erodible? low density *in situ*?
- Past landsliding? stability of extremely weathered materials in cuts?

## 3.2 VOLCANIC ROCKS (INTRUSIVE AND FLOW)

The common rocks in this group range from basalt (basic) through andesite, dacite, trachyte, to rhyolite (acidic). Basalt is the most common. All are formed from molten magma and are very fine grained, usually very strong to extremely strong when fresh. In this fresh condition the rocks generally are also very durable and are used commonly as sources of materials for filters, concrete aggregates, rockfill and road base courses. However volcanic rocks, particularly basalts and andesites, often show subtle alteration effects, which in some cases render them unsuitable for some or all of these purposes. This matter will be discussed further in Section 3.2.3. Also most volcanic rocks have initially contained some glass. In rocks of Mesozoic age and older the glass has usually "devitrified" or crystallized. However, in rocks of Tertiary and younger age the glass is usually still present today and, if the rock is used as concrete aggregate, it may react with alkalis in the cement and cause the concrete to deteriorate (see also Section 3.2.6).

The shape and other field characteristics of a body of volcanic rock depends upon the circumstances in which it solidified, i.e. as a plug, dyke, sill or flow.

### 3.2.1 *Intrusive plugs, dykes and sills*

In these types of bodies the magma has been confined within other rocks (or soils), and has flowed against them and eventually solidified against them. As a result of this mode of formation, any of the following characteristics shown in Figure 3.2 are commonly seen:

(a) The host-rock (or soil) close to the contacts may be stronger and more durable than elsewhere due to being subjected to very high temperatures;
(b) The intrusive rock has "chilled" margins, i.e. it is extremely fine-grained or even glassy, close to its contacts, due to a faster rate of cooling than in the interior of the mass;

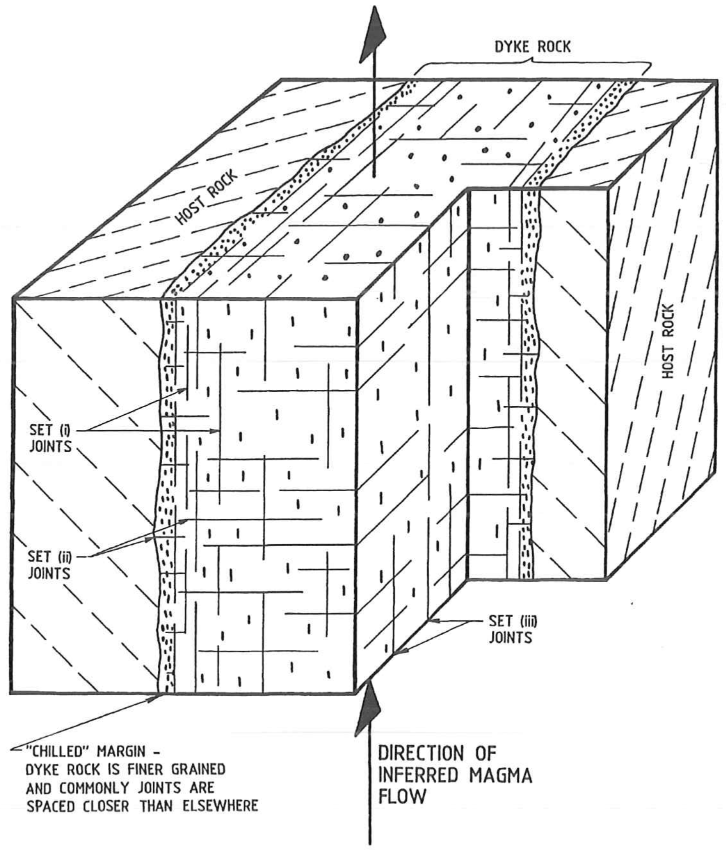



Figure 3.2. Some features commonly seen in dyke of intrusive rock.

(c) The intrusive rock has developed a "planar" foliation parallel to its contacts and, within this, a lineation, or linear arrangement of mineral grains, parallel to the direction of flow during intrusion;
(d) Joints in the intrusive body occur in at least 3 sets, as shown on Figure 3.2. Set (i) joints are parallel to the contacts (and to the foliation). Set (ii) joints are normal to the lineation i.e. to the direction of magma flow and also to the contacts. Set (iii) joints are normal to the contacts and parallel to the lineation direction. The joints in all sets commonly show extension characteristics (i.e. rough or plumose surfaces) and are either slightly open or infilled with secondary minerals including calcite and zeolite minerals. It can be inferred that shrinkage was an important factor in their formation, although extension during viscous flow seems a likely initiating factor for Set (ii).

It is sometimes found that both the host and intrusive rock are sheared or crushed, along and near the contact zone. In some cases this appears to be as a result of viscous drag, but more generally tectonically induced movements after solidification appear to be the likely cause.

In some cases, open joints in the body and contact zones may render the intrusive mass highly permeable. If continuous, such permeable masses represent potential leakage zones beneath dams or from storages.



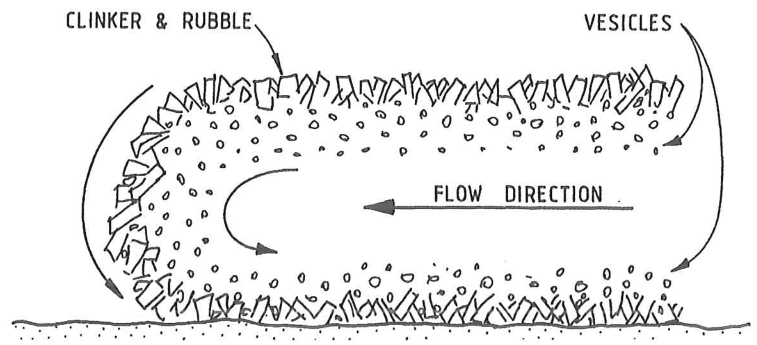



Figure 3.3. Diagrammatic longitudinal section through flowing lava showing the "caterpillar track" mechanism which results in upper and lower layers of vesicular lava and in some cases clinker or breccia.

### 3.2.2 *Flows*

Lava "flows" are bodies which have been formed by molten lava which has been extruded at the ground surface or the sea floor and has flowed over a pre-existing surface of rock, soil or sediment.

#### 3.2.2.1 *Flows on land*

Structures developed in lava flows on land are described in some detail in Hess and Poldervaart (1967), Francis (1976) and Bell (1983b). The flows move forward in a manner similar to that of a caterpillar tractor, as shown in Figure 3.3. The lava near the exposed, upper surface of the flow usually develops many small holes or vesicules, formed by bubbles of expanding gases becoming trapped as the magma solidifies. The upper surfaces of some flows develop an extremely rough, fragmented structure, comprising sharp, irregular fragments up to 150 mm across, termed clinker. Bell reports that clinker layers several million years old and buried at 500 m to 1000 m depths in Hawaii show little or no sign of compaction and are highly permeable. As the flow (Figure 3.3) moves forward the clinker-covered surface and vesicular layer are carried forward, deposited over the front and eventually buried beneath the flow. Thus the solidified flow comprises an inner layer of massive rock sandwiched between two layers of clinker and vesicular rock which are highly permeable.

Other lava flows develop a hummocky and sometimes twisted ropy structure at their upper surfaces, due to viscous drag on the surface crust while this is still plastic.

Where successive lava flows have occurred to form a continuous layered sequence, the individual flows and their boundaries may be distinguishable by the following:

- The development of a weathered or soil profile on the upper surface of a flow which was exposed to weathering for some time before the next flow occurred;
- The presence of chilled and vesicular zones, or clinkered, brecciated or ropy zones near flow boundaries.

At Foz do Areia dam in Brazil, breccia (clinker) zones at the boundaries of basalt flows were locally weathered and generally highly permeable (Figure 3.4). Treatment of these zones by excavation, dental concrete and grouting is described by Pinto et al. (1985).

It is common for lava flows to be interbedded with pyroclastic materials (ash and lava fragments) or tuff and agglomerate in rock sequences derived from volcanic explosions. Also flows may occur interbedded with alluvial or other sediments.

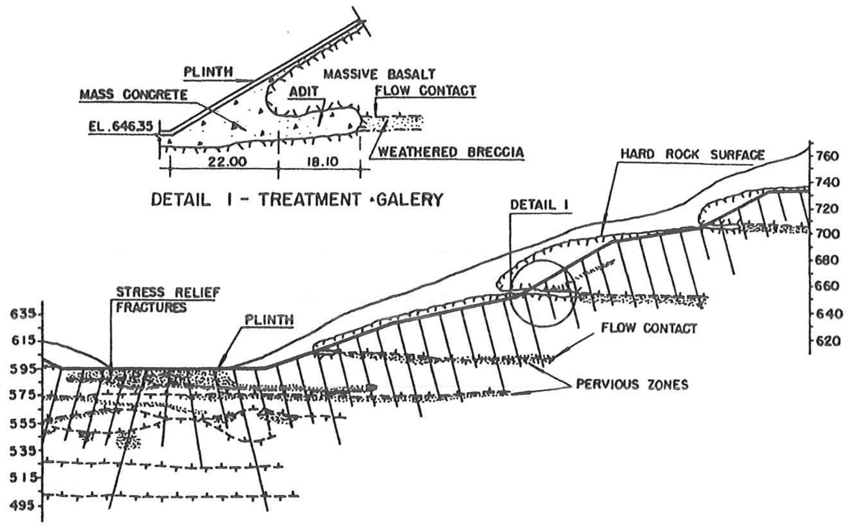



Figure 3.4. Geological section along Plinth (right bank) at Foz Do Areia Dam (Pinto et al., 1985).

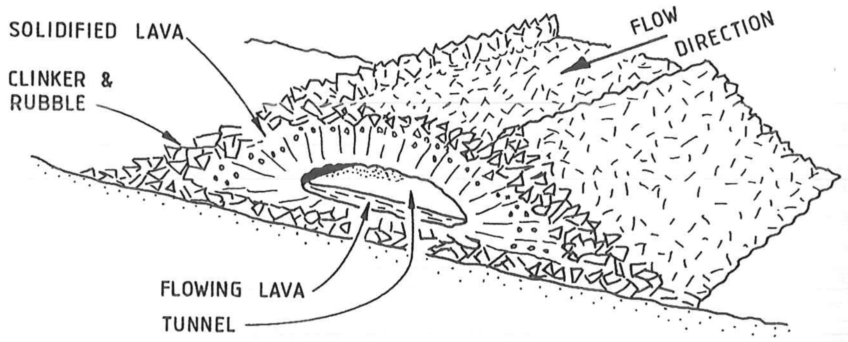



Figure 3.5. Perspective view of, and cross section through a lava flow, showing a lava tunnel developed when the lava flows forward faster than the supply.

Thick lava flows sometimes contain "lava tunnels", which are circular, ovoid or lenticular in cross section, up to 20 m across and some kilometres in length. These are formed when the supply of lava to the flow is exhausted, and the internal lava "stream" (Figure 3.5) drains away. Figure 3.6 shows a small lava tunnel exposed in basalt forming the wall of a 30 m diameter shaft at Hoppers Crossing, Victoria, Australia.

Most lava flows show a hexagonal columnar joint pattern, with the columns being interrupted by near-planar or saucer-shaped cross joints. These joints have developed as a result of shrinkage on cooling and are usually either slightly open or filled with secondary minerals or alteration products. Figure 3.7 shows columnar-jointed rhyolite exposed during construction at High Island dam, in Hong Kong. The rhyolite has been intruded by a younger, darker rock forming a dyke up to 1.5 m thick.

In some flows there are closely spaced joints parallel and near to the margins, apparently caused by shearing associated with viscous flow. Where columnar joints in lava flows are open, e.g. due to mechanical weathering close to steep valley sides, the rock mass permeability can be high.



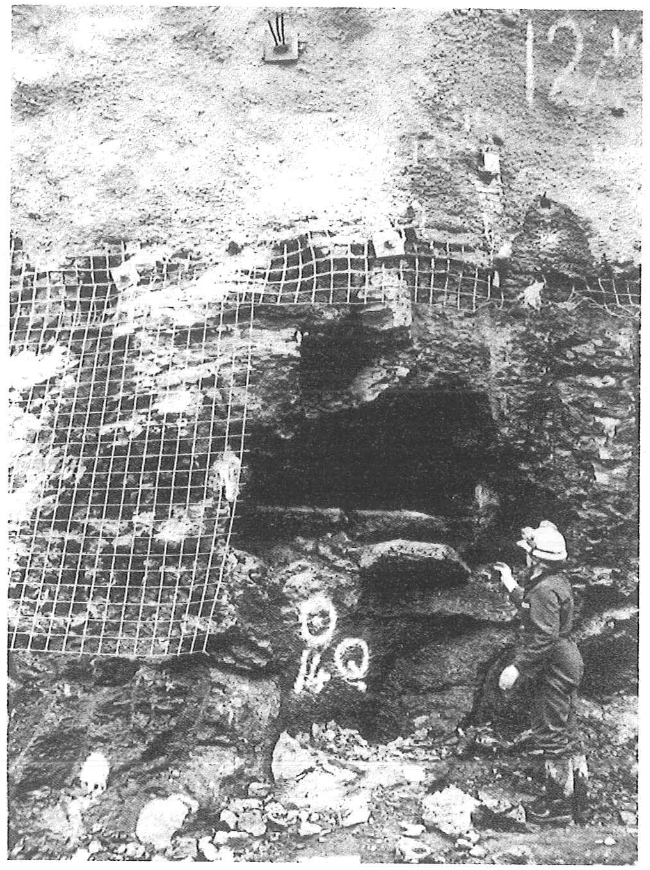


Figure 3.6. Lava tunnel in basalt, Hoppers Crossing, Victoria, Australia.

Figure 3.7. Columnar joint pattern in rhyolite, and younger intrusive dyke. High Island Dam, Hong Kong.

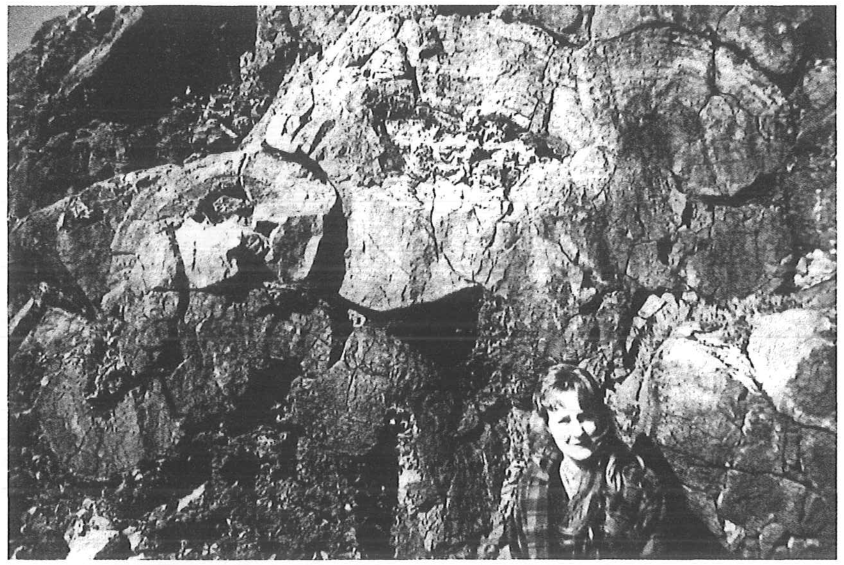


Figure 3.8. Pillow structure in basalt, Pembrokeshire, U.K.

As stated at the start of this chapter, fresh volcanic rocks are used widely as construction materials. However, columnar jointed rock can be difficult to quarry, especially when the columns are almost vertical. Blastholes drilled vertically tend to jam in the open joints and explosive gases vent into them. The columns tend to topple over rather than fragment, resulting in a poorly graded (one-sized) quarry-run product. If the column diameter is large (i.e. more than 1 metre) then the product may be too large for the crusher. Regardless of the column size the poorly graded quarry product does not make good rockfill because it is difficult to compact and large voids remain between the rock blocks after compaction. This rockfill often has relatively low modulus and because of its large voids, requires special attention to filter design.

#### 3.2.2.2 *Undersea flows*

Lava which has been extruded from the sea-bed is often in the form of distorted globular masses up to 3 m long known as pillows (Figure 3.8).

The pillows may be welded together, separated by fine cracks, or separated by sedimentary detritus derived from the lava. The long axes of the pillows are generally roughly parallel to the boundaries of the "flow". Bell (1983b) notes that joints, vesicules, and phenocrysts in the pillows are in some cases arranged radially. The joint pattern and the irregular shapes of the cracks at the pillow boundaries produce an overall fracture pattern that might at first appear to be "random".

### 3.2.3 *Alteration of volcanic rocks*

Fresh volcanic rocks are composed of minute, strong, tough mineral crystals, generally arranged in an extremely dense, interlocking manner. This structure results in negligible porosity and great strength and durability within the lifetime of engineering structures. However, as stated earlier, these rocks are often found to have been altered, probably during the late stages of solidification. In the altered rocks some of the crystals are wholly or partly changed to secondary minerals, including serpentine, calcite, chlorite, zeolites and clay



minerals. These minerals are weaker and in some cases larger in volume than the original minerals, resulting in microcracking, increase in porosity and weakening of the rock. The altered rock is usually greenish in colour.

Where these effects are pronounced, and particularly if the clay mineral montmorillonite is produced, the altered rock is obviously much weaker than the original rock and is likely to deteriorate or even disintegrate on exposure to air or immersion in water. However if the alteration effects are relatively minor, e.g. only the margins of the original crystals are changed to secondary minerals, then the visual appearance of the rock substance may be little changed from that of fresh, unaltered rock. The altered rock will usually appear a little dull and have a slightly higher porosity and absorption than the fresh rock. Rocks such as this, showing only very minor and subtle alteration effects, often deteriorate rapidly in pavements, are likely to be unsuitable for crushing to produce filters and may prove to be unsuitable for concrete aggregate and rockfill. Because of this, before adopting volcanic rocks for use as construction materials, they should always be subjected to very thorough checking, by local past performance and field observations, petrographic analysis and laboratory tests. For further details readers are referred to Shayan and Van Atta (1986), Van Atta and Ludowise (1976), Cole and Beresford (1976), Cole and Sandy (1980) and Hosking and Tubey (1969).

As the alteration is caused by hot water and gases which move through permeable features such as joints and highly vesicular zones, the more altered rock is usually located along and adjacent to such features. Quite commonly, secondary minerals occur as veins, seams or irregular masses filling previously gaping joints or voids. Such features, particularly where of large extent and composed of clay, serpentine or chlorite, represent significant rock mass defects with low shear strength.

### 3.2.4 *Weathering of volcanic rocks*

Although all unaltered volcanic rocks are highly durable within the life-span of normal engineering structures, the more basic varieties, particularly basalt, are quite susceptible to chemical weathering in a geological time frame. The distribution of weathered materials in volcanic rocks is governed by the distribution of any previously altered material, as well as by the pattern of joints and vesicular zones.

When extremely weathered, all volcanic rocks are clayey soils in the engineering sense. The acidic types tend to produce low or medium plasticity clays and the basic types high plasticity clays. Basalts commonly display spheroidal weathering profiles, with spheroids of fresh to distinctly weathered basalt surrounded by extremely weathered basalt, which is clay of high plasticity. Extremely weathered basalts and the surface residual soils developed on them are usually highly expansive and fissured.

### 3.2.5 *Landsliding on slopes underlain by weathered basalt*

The presence of a well developed pattern of slickensided fissures causes the shear strength of the mass to be significantly lower than that of the intact material. Slopes steeper than about 10 degrees which are underlain by such materials often show geomorphological evidence of past or current landsliding.

Such landsliding occurs commonly at the steep margins of plateaus or hills capped by basalt flows which overlie old weathered land surfaces, as shown in Figure 3.9. The sliding occurs in some cases simply as a result of over-steepening of the hillside by erosion, and in others due to pressure from groundwater exiting from a permeable zone beneath the extremely weathered basalt. The permeable zone may be jointed, less weathered basalt as in Figure 3.9, or alluvial sands or gravels on the old buried land surface. Examples are given in Fell (1992).

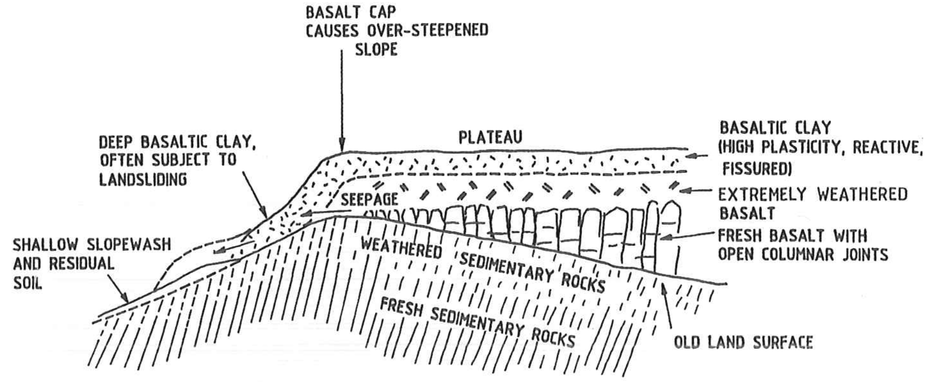



Figure 3.9. Typical profile through margin of basalt plateau, showing conditions which lead to slope instability.

Landsliding in fissured, extremely weathered basalt in Victoria, Australia is described by MacGregor et al. (1990).

### 3.2.6 *Alkali-aggregate reaction*

As discussed in Section 3.2 above, most volcanic rocks of Tertiary and younger ages contain appreciable amounts of glass, which may react with alkalis in Portland cement if the materials are used as aggregates in concrete. In older rocks, much or all of the glassy materials have usually developed a very fine crystalline structure and are less likely to be reactive. Zeolite minerals which occur in many volcanic rocks may also react with alkalis in cement.

Guillott (1975, 1986) and McConnell et al. (1950) describe observed effects of concrete expansion due to alkali aggregate reaction and discuss the mechanisms involved. Stark and De Puy (1987) describe observations and tests on affected concrete at five dams in USA. Cole and Horswill (1988) describe the deleterious effects and remedial work carried out at Val de la Mare dam, in Jersey, Channel Islands.

Shayan (1987) and Carse and Dux (1988) discuss some limitations of chemical and mortar bar tests for the prediction of the actual performance of aggregates in concrete.

The deleterious effects can be largely avoided by the use of low-alkali cement or pozzolanic additives in the concrete.

If volcanic rock is to be used as aggregate careful checking for possible alkali-aggregate activity is advisable, particularly if low-alkali cement is not available. Studies usually include checking of past performance, petrographic examination (ASTM, 1974a), and laboratory testing including the Quick Chemical Test (ASTM, 1974b or Standards Association of Australia, 1974a), the Gel-Pat Test (Building Research Station, 1958) and the Mortar Bar Test (ASTM, 1974c or Standards Association of Australia, 1974b).

### 3.2.7 *Volcanic rocks (intrusive and flow) check list of questions*

- Vesicular zones?
- "Clinker" or "breccia" zones?
- Lava tunnels?
- Old weathered/soil profiles?
- High mass permeability?
- Interbedded pyroclastic or sedimentary materials?



- Columnar joint pattern?
- Toppling failure?
- Difficulties in blasthole drilling?
- Poor fragmentation during blasting?
- Irregular joint pattern and "pillow" structure?
- Alteration effects – secondary minerals?
- Fresh, extremely strong boulders in extremely weathered materials (high plasticity clay)?
- Very high plasticity soils, expansive, fissured?
- Unstable slopes?
- Alkali-aggregate reaction?

## 3.3 PYROCLASTICS

Pyroclastic or "fire-broken" deposits are those which have been formed by the accummulation of solid fragments of volcanic rock, shot into the air during volcanic eruptions. The rock fragments include dense, solidified lava, highly vesicular lava (termed scoria) and extremely vesicular lava (termed pumice). Pumice is formed only from acidic lavas (e.g. rhyolite) and is so porous that it will float on water. Francis (1976) provides a detailed account of the ways in which pyroclastic materials are formed, based mainly on historical accounts of modern eruptions. Prebble (1983) describes the pyroclastic deposits of the Taupo Volcanic Zone in New Zealand and the difficulties they present in dam and canal engineering.

### 3.3.1 *Variability of pyroclastic materials and masses*

Pyroclastic deposits are characterized by extreme variability in engineering properties over short distances laterally and vertically. They range from extremely low density "collapsing" type soils to extremely strong rocks. This wide range in properties results from differences between the ways in which they were initially deposited and also from the ways they have been modified since deposition.

There are four main types of deposit, based on initial mode of deposition:

(a) Air fall deposits in which the fragments have simply been shot up into the air and fallen down again. Where they have "soil" properties such deposits are termed ash (sand sizes and smaller), or lapilli and bombs (gravel sizes and larger). Where welded, compacted or cemented to form rocks, they are termed tuff (sand sizes and smaller) or agglomerate (gravel sizes and larger in a matrix of ash or tuff).

(b) *Water-sorted deposits* in which the fragments fall into the sea or a lake and become intermixed and interbedded with marine or lake deposits. These also may be "soils" or rocks depending upon their subsequent history.

(c) *Air-flow, or "nuées ardentes", deposits* in which the fragments are white-hot and mixed with large volumes of hot gases, to form fluidized mixtures which can travel large distances across the countryside at speeds of probably several hundreds of kilometres per hour. The resulting materials, known as ignimbrites, range from extremely low density soils with void ratios as high as 5 (Prebble, 1983) to extremely strong rocks. The latter are formed when the white-hot fragments become welded together to form rocks almost indistinguishable from solidified lavas. These rocks are called welded ignimbrites or welded tuffs.

(d) Hot avalanche deposits which are formed by the gravitational breakup and collapse of molten lavas on steep slopes. The deposits comprise loosely packed but partly welded boulders, often showing prismatic fracture patterns which indicates that they were chilled rapidly after deposition (Francis, 1976).

Following or during their initial deposition, any of these types of deposit may be modified greatly by any or all of the following, which prevail in active volcanic environments:

- Faulting;
- Intrusion of further igneous material in plugs or dykes;
- Lava flows;
- Intense thunderstorm activity which causes erosion and redeposition, landsliding and mudflows;
- Hydrothermal alteration and chemical weathering.

The alteration and weathering produce clay minerals which may include montmorillonite, noted for its low shear strength, nontronite and allophane and halloysite which, in some cases, are highly sensitive.

The air-flow and air-fall deposits have commonly been deposited on old land surfaces with variable relief and covered by residual soil and weathered rock profiles. Prebble (1983) notes that such deposition in the Taupo Volcanic Zone of New Zealand resulted in permeable sand and gravel sized materials burying weak and relatively impermeable residual clays in old valleys. Subsequent cycles of deposition and near-surface weathering have resulted in a "valley-upon-valley" sequence of aquifers, interlayered with clayey aquicludes which tend to be sensitive and collapsible (see Figure 3.10).

Pyroclastic materials are also found in near-vertical pipes or necks, called diatremes. The Kimberley diamond pipe in South Africa and the Prospect Diatreme in Sydney, New South Wales (Herbert, 1983) are examples. The materials in the diatremes comprise angular fragments of volcanic rock plus fragments of the underlying and surrounding rocks. It seems likely that diatremes have been formed by explosive volcanic eruptions.

All of the above processes result in the extreme variability found in modern volcanic deposits. In geologically old deposits, which have been deeply buried by later sedimentation and folded, faulted and uplifted, new defects and variabilities are introduced, but the effects of compaction and consolidation can cause the strength contrasts between the various pyroclastic substances to be greatly reduced.

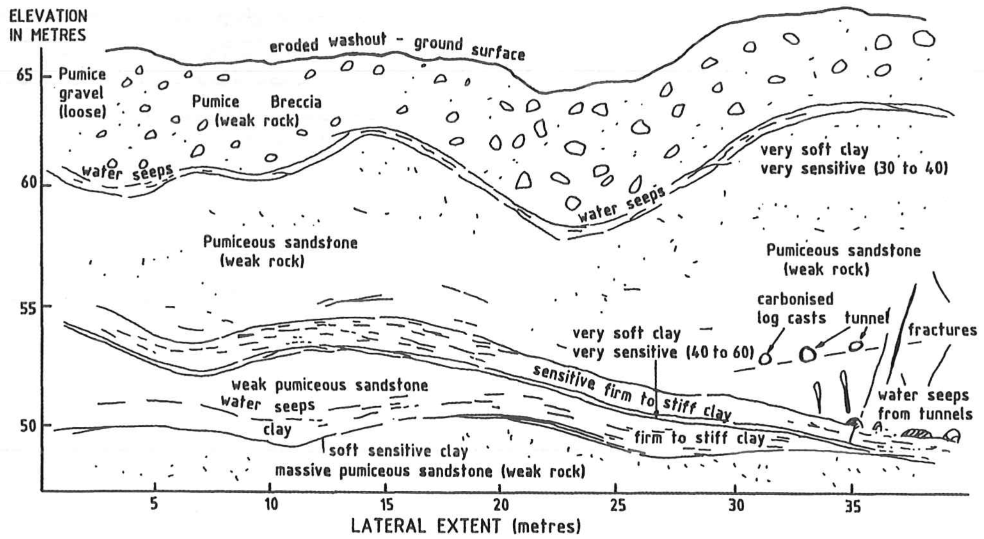



Figure 3.10. Exposure of pyroclastic materials in the collapsed area of Ruahihi Canal, New Zealand (Prebble, 1983).



### 3.3.2 *Particular construction issues in pyroclastics*

Prebble (1983), Jones (1988) and Oborn (1988) describe problems which were encountered at the Ruahihi and Wheao hydroelectric projects, constructed in the Taupo Volcanic Zone. Headrace canals for each project were constructed by cut and fill methods, on and through highly variable ash and ignimbrite deposits. Both canals failed by piping and subsequent collapse during early operation, apparently due to the high erodibility of some of the soils, both *in situ* and when compacted, and to their brittle, non-healing nature which enabled the development of erosion tunnels. Oborn (1988) suggests that some of the soils at Ruahihi were probably dispersive and that accelerated rates of settlement after canal filling may have been due to the collapse of very low density soils on saturation after loading. At the Wheao failure area, erodible ash soils were located above very high strength welded ignimbrite with a columnar joint pattern. Near the upper surface of the ignimbrite, the joints were "bridged" by infill soils from above, but below this they were open as much as 50 mm. This feature was apparently missed during the construction stage cleanup. During operation, erodible ash soils were washed into these gaping joints, close to the penstock intake structure as shown on Figure 3.11, taken from Jones (1988). Prebble (1983) and Oborn (1988). Note also that the extreme sensitivity (up to about 60) of some of the alteration products (allophane and halloysite clays) caused problems during construction of the canals. Prebble predicts that these soils could collapse and liquefy when disturbed by earthquake loading or changed groundwater levels.

Jacquet (1990) describes the results of comprehensive laboratory tests on andesitic ash soils from seven sites in New Zealand. The soils contained high proportions of allophane or halloysite and all classified as MH in the Unified System. Sensitivities ranged from 5 to 55. Jacquet concluded that the sensitivity was associated with irreversible rupture of the structural fabric of the soils and was not directly related to the clay mineralogy or classification characteristics of the soils.

Not all weathered pyroclastic materials are sensitive. In weathered agglomerates at the site of Sirinumu Dam in Papua New Guinea the matrix soils, which are clays of medium to high plasticity with 40 to 50 percent moisture content, are very resistant to erosion. The clay mineral types include halloysite, kaolin and allophane.

Rouse (1990) describes tropically weathered andesitic and dacitic ash soils from Dominica, West Indies, which occur on generally stable slopes of 30° to more than 50°. The soils are mainly allophane and halloysite clays, with very high residual friction angles (most between 25° and 35°).

Some unweathered non-welded ash and weakly-welded ignimbrite materials can be used as sand and gravel sized embankment filling, the weakly welded materials breaking down readily during compaction. However based on the experience at the 60 m high Matahina rockfill dam in New Zealand (Sherard, 1973) it is suggested that such materials should not be used for filter zones unless it can be shown that they will remain cohesionless in the long term. At Matahina weakly welded, partly weathered ignimbrite was compacted to form "transition" zones between the impervious core and rockfill zones. Subsequent excavation through the compacted ignimbrite showed it to have developed appreciable cohesion and it appeared to behave like a very low strength rock. This strength developed due to the interlocking of needle-shaped particles of glass. This cohesive, brittle behaviour was an important contributing factor to the piping incident which occurred during the first filling of the reservoir in January 1967. The main cause of this incident, which resulted in the loss of more than 100 $m^3$ of core and transition materials into the rockfill, was the failure to remove a 1.8 m projecting bench from the steeply sloping foundation. This projection caused cracks to develop in the core and transition zones as the embankment settled. Other factors, as described by Sherard (1972, 1973) and Gillon and Newton (1988), included the possible reinforcement of the transition zones by

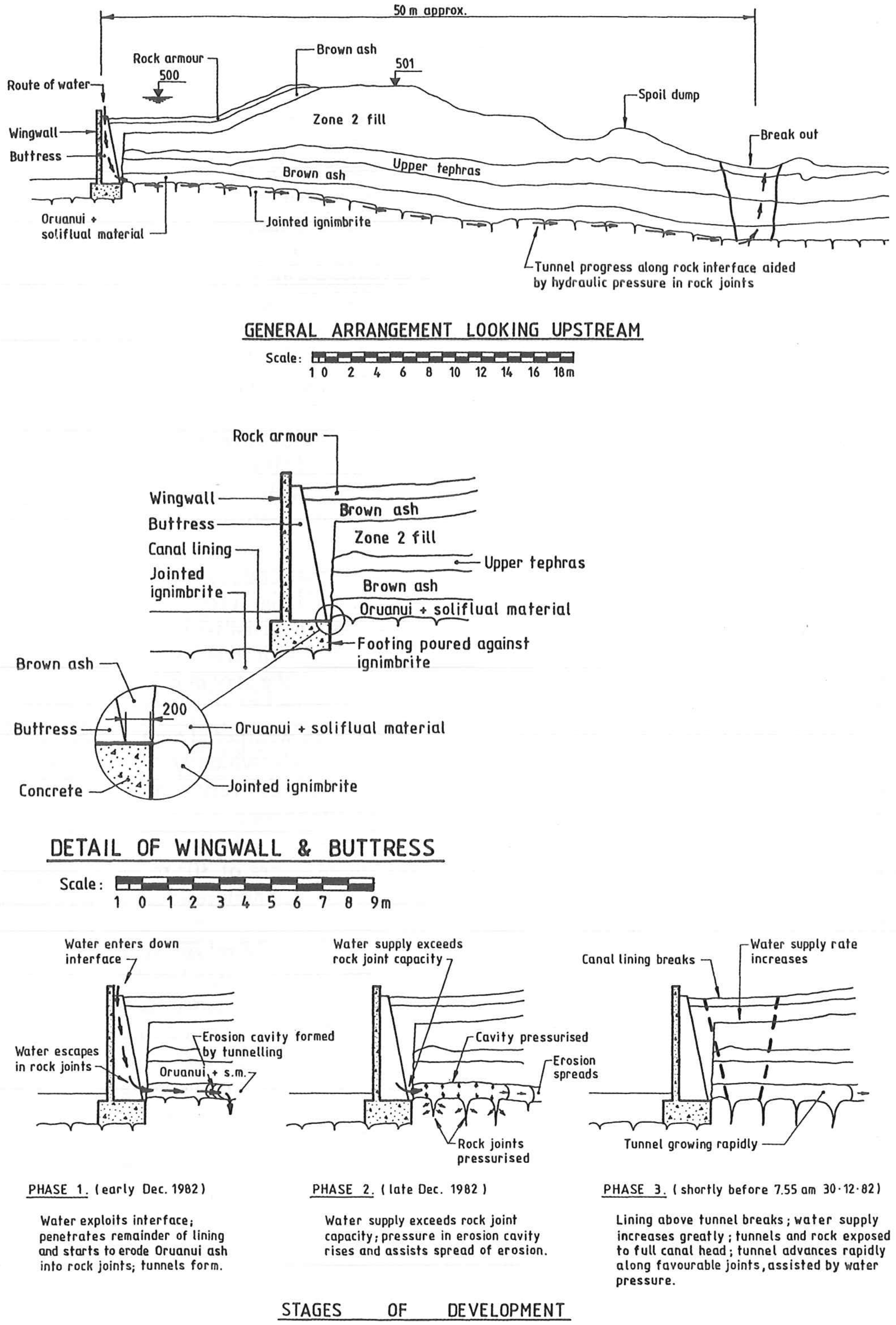



Figure 3.11. Wheao Canal failure. Diagrams showing the general arrangement and how the failures developed (Jones, 1988).



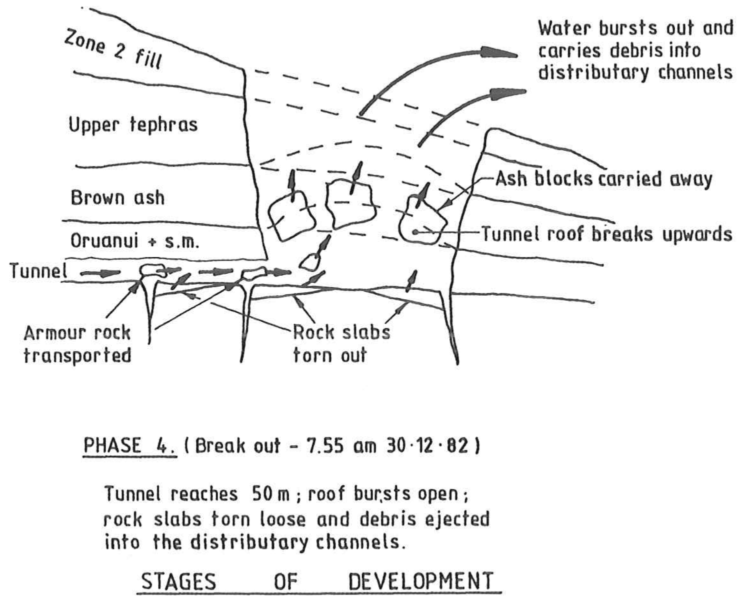



Figure 3.11. (Continued).

means of cement grout, the dispersive nature of the core material and large voids in the rockfill shoulders which were formed by poorly graded very strong welded ignimbrite, quarried from a columnar-jointed mass. Most blocks in the rockfill are of blocky shape and in the range 300 mm to 600 mm (Section 3.2.2.1).

As with lavas, pyroclastic materials, especially those of Tertiary or younger ages, contain glassy materials which may react with alkalis in Portland cement. They should therefore be tested thoroughly before use as aggregates in concrete (Section 3.2.6).

### 3.3.3 *Pyroclastic materials – check list of questions*

- Extreme variability?
- Very low *in situ* densities – collapse type behaviour?
- High *in situ* permeability?
- Brittle *in situ* and when compacted?
- Highly erodible *in situ* and when compacted?
- Highly to extremely sensitive zones?
- Complex groundwater distribution?
- Welded rocks: gaping joints?
- Columnar jointed welded rocks: poorly graded rockfill, quarrying problems?
- Interbedded lavas?
- Intrusive dykes, sills or plugs?
- Alkali-aggregate reaction?

## 3.4 SCHISTOSE ROCKS

Included in this group are those metamorphic rocks, e.g. slate, phyllite and schist which have developed a pronounced cleavage or planar foliation. The cleavage or foliation results from the parallel arrangement of platy minerals, commonly clays, muscovite, biotite, chlorite and sericite. Also often present and in parallel arrangement, are tabular or elongate clusters of other minerals, usually quartz and felspars and occasionally amphiboles.

Although the foliation is referred to as "planar", the foliae or layers are commonly folded. The folds can range in amplitude and wave-length from microscopic up to hundreds of metres. Small-scale folds, which cause the surfaces of hand-specimens of schist to appear rough or corrugated, are called crenulations.

In some schists the foliation has been so tightly and irregularly folded as to give a contorted appearance. Such rock is called "knotted schist".

Slate, phyllite and most mica schists have been formed by the regional metamorphism of fine grained sedimentary rocks (mudstones or siltstones). This has involved relatively high temperatures and directed pressure over long periods of geological time. Under these conditions the clay minerals present in the original rocks have changed partly or wholly to mica minerals, usually muscovite, chlorite and biotite. These minerals have become aligned normal to the direction of the maximum compressive stress. The proportion of these new minerals is least in slate and greatest in schist.

Greenschists, which are well foliated rocks containing a large proportion of chlorite and other green minerals, have been formed by the regional metamorphism of basic igneous rocks (e.g. basalt and gabbro).

Schists can be formed also as a result of shear stresses applied over long periods of geological time to igneous rocks or to "high-grade" metamorphic rocks, e.g. gneisses. This process is known as retrograde metamorphism and usually produces schist containing abundant sericite and/or chlorite. These are weak minerals and so the rocks produced by this process tend to be weaker than other schists.

### 3.4.1 *Properties of fresh schistose rock substances*

Most schistose rocks have dry strengths in the very weak to medium strong range (see Table 2.4). Schists containing abundant quartz are generally medium strong or strong, while greenschists which are rich in chlorite are generally very weak.

The most significant engineering characteristic of schistose rocks is their pronounced anisotropy, caused by the cleavage or foliation. Figure 3.12 taken from Trudinger (1973) shows the results of unconfined compressive tests on fresh schist samples from Kangaroo Creek Dam, South Australia. This schist is stronger than most; it contains generally about 40% quartz and felspar and about 60% sericite and chlorite. It is foliated but not exceptionally fissile. It can be seen that the strengths recorded for samples loaded at about 45° to the foliation were about one third of those for samples loaded at right angles, i.e. the anisotropy index of this schist in unconfined compression is about 3.

When tested by the point load method, i.e. in induced tension, the schist at Kangaroo Creek shows anisotropy indices ranging typically from 5 to 10. Failure along the foliation surfaces in this test is by tensile splitting, rather than in shear.

Most other schists, slates and phyllites show similar anisotropic properties (Donath, 1961). It is clear that foliation angles in relation to loading directions should always be carefully recorded during tests and reported with the results.

In weak or very weak schists (often those rich in chlorite) the effective angle of friction along foliation surfaces can be low. Landsliding is prevalent in areas underlain by these rocks.

In knotted schists the foliation surfaces are often so contorted that shearing or splitting along near-planar surfaces is not possible. As a result, knotted schists are usually appreciably stronger than those in which the foliation surfaces are near-planar.

Figure 3.12 also indicates that the schist at Kangaroo Creek Dam showed a 25–65% reduction in strength, after soaking in water for 1–2 weeks. The greatest strength reduction



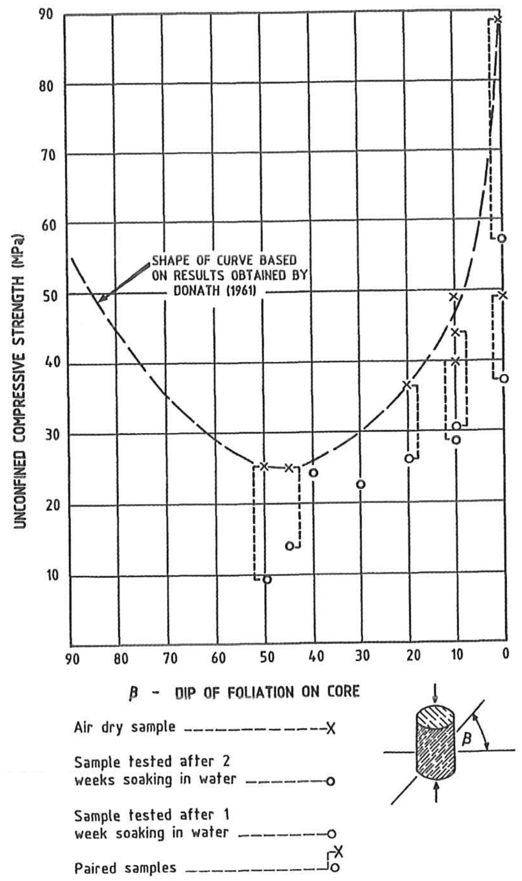



Figure 3.12. Variation of unconfined compressive strength of schist with angle between the foliation planes and applied loads (Trudinger, 1973).

occurred in the samples loaded at about 45° to the foliation. This result is typical of schistose rocks.

### 3.4.2 *Weathered products and profiles developed in schistose rock*

Schistose rocks vary widely in their susceptibility to chemical weathering. Varieties rich in quartz are very resistant and, at the other extreme, rocks rich in clay minerals or chlorite are very susceptible. Because of this, no typical weathered profile exists for all schistose rocks. However many schistose rock masses consist of layers (parallel to the foliation) of varying susceptibilities to weathering. Thus weathering often tends to exaggerate the strength anisotrophy of such masses and the upper surface of fresh rock often has a deeply slotted or serrated shape, as shown in Figure 3.13.

Schists which are rich in micaceous minerals (biotite, muscovite or chlorite) tend to form micaceous silty or clayey soils when extremely weathered. The silty varieties are often of low *in situ* density and are highly erodible by water or wind. Also they tend to be hydrophobic, making dust control difficult on construction sites.

Even when fresh or only partly weathered, the more micaceous, fissile schists usually produce much dust due to abrasion during handling and trafficking.

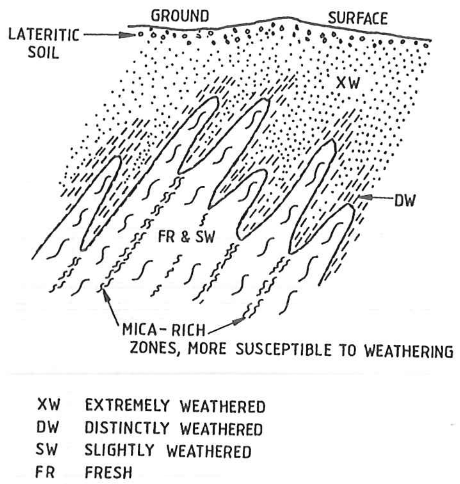



Figure 3.13. Weathered profile developed on schistose rock with steeply dipping foliation.

### 3.4.3 *Suitability of schistose rocks for use as filter materials, concrete aggregates and pavement materials*

Schistose rocks are generally unsuitable for any of these purposes due to the very flaky shapes of the crushed materials and inadequate strengths of the particles. Kammer and Carlson (1941) describe the unsuccessful use of phyllite as aggregate for concrete in a hydro-electric plant. Silicates in the phyllite reacted with alkalis in the cement to cause expansion and disruption of the concrete. The strongest, most siliceous schists have been used successfully in base courses of pavements, where more suitable materials have not been available.

### 3.4.4 *Suitability of schistose rocks for use as rockfill*

Despite their tendency to produce very platy block shapes, schistose rocks have been used successfully as rockfill on several dams up to 80 m high. At Kanmantoo Mine in South Australia a 28 m high rockfill dam with a thin sloping earth core was built in 1971 for the storage of tailings and water for use at the mine (Stapledon et al., 1978). The rockfill was mainly very weak quartz-biotite schist, the waste rock from the mine. It was placed in 0.6–0.9 m thick layers and compacted dry with a Caterpillar D8 tractor and by trafficking by the 50-tonne dump trucks. Post-construction crest settlements 5.5 years after completion ranged from 0.3% to 0.7% of the embankment height.

The successful use of schist as rockfill at Kangaroo Creek Dam has been described by Good (1976) and Trudinger (1973). Trudinger describes in some detail the construction procedures, the behaviour of the rock and the fill densities obtained. The concrete faced dam as built initially was 60 m high. Zone 3, which contained about 300,000 $m^3$ of the 443,000 $m^3$ total rock in the embankment, was built mainly of weak to medium strong, slightly weathered schist compacted in 1 m layers. The maximum specified block size was 1 m, but many blocks were longer than this due to the very platy shapes obtained during quarrying (Figure 3.14). Because the schist suffered a large strength loss on saturation (Figure 3.12) the rock was heavily watered during placement. During compaction by 4 passes of a 10-tonne vibrating roller, the uppermost 50–300 mm of most layers were crushed into a gravelly sandy silt (Figure 3.15). These layers formed thin, lower permeability barriers but were readily penetrated by the basal rocks of the next layer and hence they did not form weak or compressible zones in the dam.



Figure 3.14. Schist being quarried in the spillway excavation at Kangaroo Creek Dam. Note the platy shapes of the rock fragments.

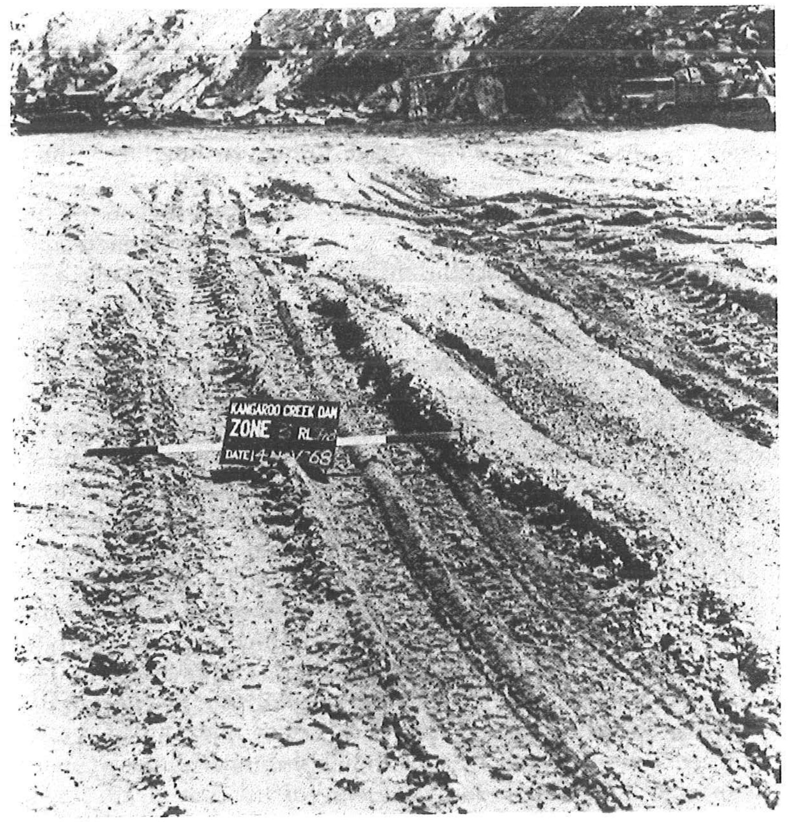



Figure 3.15. Slurry (Gravelly sandy silt) formed on the top of schist rockfill layer as a result of weathering and compaction by 4 passes of a 10-tonne vibrating roller (Trudinger, 1973).

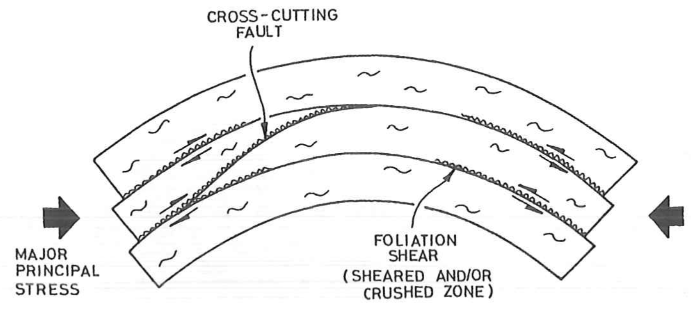



Figure 3.16. Probable way in which foliation shears and cross-cutting faults are formed in some schistose rocks.

After 20 years of operation the dam height was increased to 64 m by placing a rock-filled reinforced concrete trough across its crest.

In 2003, thirty four years after its initial completion, the maximum creep settlement at the original crest level was 184 mm, which is 0.3% of the initial height and 0.29% of the revised height.

### 3.4.5 *Structural defects of particular significance in schistose rocks*

Three defect types are discussed below.

#### 3.4.5.1 *Minor faults developed parallel and at acute angles to the foliation*

Schists commonly contain minor faults (narrow, sheared zones or crushed seams, or both) parallel to the foliation. Deere (1973) refers to these features as foliation shears. In folded schists, the foliation shears have probably been formed by inter-layer slip, as shown on Figure 3.16.

Also present in many folded schists are similar "shears" cutting across the foliation at acute angles, generally less than 20° (Figure 3.16). In some cases these are thrust faults. Residual shear strengths of both foliation shears and cross-cutting faults have been found in laboratory tests (i.e. excluding the effects of large scale roughness), to lie in the range 7° to 15°. Such defects commonly form the initiating failure surfaces of landslides in schistose rocks (Figure 3.17) and may provide potential sliding surfaces into spillway or foundation excavations, or within the foundations of an embankment dam.

As these features are often only 50 mm or so in thickness, they can escape detection during site investigations unless the investigator sets out to look for them, using appropriate techniques, e.g. well cleaned up trenches and high quality core drilling. The sheared zones can be particularly difficult to detect. In these zones the rock is more intensely foliated than elsewhere and is usually rich in chlorite and/or sericite. The sheared material is therefore appreciably weaker than the normal schist and is readily recognizable when the rock is fresh. However, in distinctly or extremely weathered exposures in which both sheared and unsheared materials are greatly weakened by weathering, it can be quite difficult to recognize the sheared zones, because the strength contrast is much reduced, and the shear-induced cleavage or foliation is similar in appearance to, and may be parallel to, the foliation in the normal schist. Stapledon (1967) describes how initial, poor quality core drilling at Kangaroo Creek (South Australia) failed to indicate the presence of foliation shears and associated infill clay seams. Discovery of these in a later exploration programme led to abandonment of a thin concrete arch design in favour of a decked rockfill dam. Paterson et al. (1983) describe how surface mapping and diamond drilling were not adequate to define the full extent and frequency of foliation shears at the site for Clyde



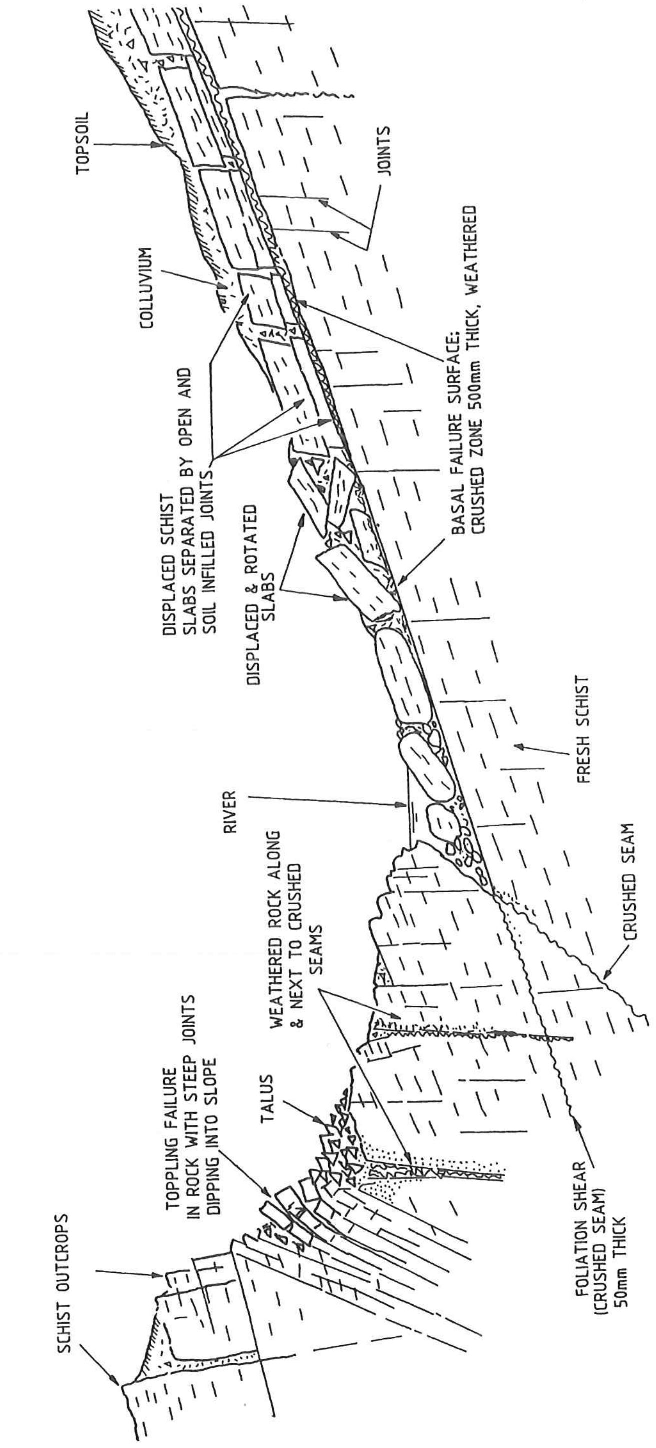



Figure 3.17. Typical valley profile in gently dipping schist affected by past landsliding along foliation shear.

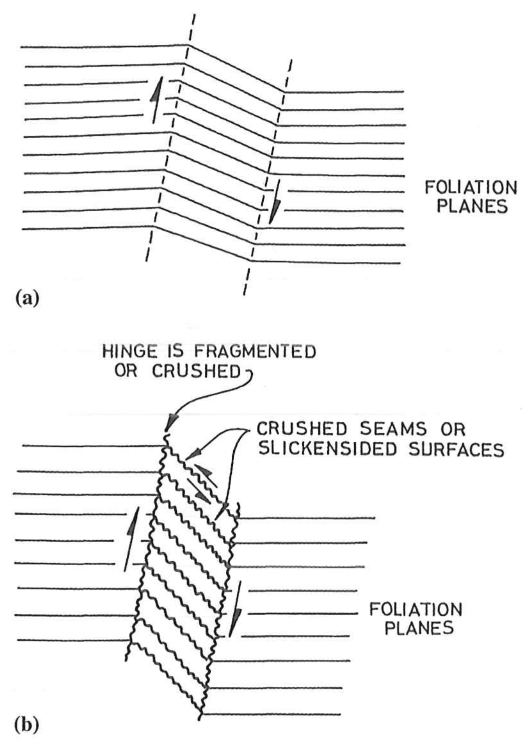



Figure 3.18. Formation and nature of kink bands.

Dam in New Zealand. The location and treatment of some of these shears during construction of the dam are discussed in Section 17.8.3.

#### 3.4.5.2 *Kink bands*

Schistose rocks often also contain "kink bands" within which the foliation layers have been displaced to form features similar to monoclinal folds but with sharp, angular hinges (Figure 3.18a).

In some cases continuous near-planar joints have developed along the hinges and the foliation layers within the band have parted to form open or infilled joints. In other cases crushed seams have developed along the hinges and the foliation layers within the band are either slickensided or partly crushed (Figure 3.18b). Such kink bands can be considered a special class of fault.

#### 3.4.5.3 *Mica-rich layers*

Some schists contain layers or zones which consist almost entirely of micaceous minerals, e.g. biotite, muscovite, sericite or chlorite. Such layers or zones are usually much weaker than the normal schist. It is good practice to consider them as individual defects of low shear strength.

### 3.4.6 *Stability of slopes formed by schistose rocks*

Landsliding is relatively common on slopes underlain by schistose rocks. Well-known examples are the Madison Canyon Rockslide in Montana, U.S.A. (Hadley, 1978), the



Downie Slide in British Columbia (Piteau et al., 1978) and the Tablachaca Slide in Peru (Arnao et al., 1984; Deere and Perez, 1985).

The landsliding usually occurs by slope failure along weathered foliation surfaces or foliation shears as shown on Figure 3.17. Bell (1976, 1982) describes how the development of a river valley in New Zealand has involved landsliding along dipping foliation surfaces.

In steeply dipping schistose rocks failure by toppling is also relatively common, the toppled slabs or columns being separated by joints or shears along the foliation direction. Riemer et al. (1988) describe a complex toppling failure at San Pablo, Peru.

Where the schistose rocks contain abundant tectonically-formed defects in other orientations, many other failure models have been recorded. However in most of these it is likely that the low shear strength of the schistose rocks along their foliation surfaces has contributed to the development of slope failure.

In long, high slopes in mountainous areas some failures of schistose rocks appear to have occurred by buckling of, and eventually shearing through, the foliation. The buckling is facilitated by the low shear strength of the foliation surfaces which allows multiple shear displacements to occur along them. Examples of this type of slope failure are discussed in Beetham et al. (1991), Riemer et al. (1988), Radbruch-Hall et al. (1976), Nemcok (1972) and Zischinski (1966, 1969).

Examples of landsliding in schists forming slopes around the reservoir of Clyde Dam in New Zealand are discussed in Gillon and Hancox (1992), Riddolls et al. (1992) and Stapledon (1995).

### 3.4.7 *Schistose rocks – check list of questions*

- Degree of anisotropy, and its effect on the project?
- Low durability in exposed faces?
- Particle shapes and strengths inadequate for filter, concrete or pavement materials?
- Suitability for use as rockfill?
- Foliation shears?
- Kink bands?
- Mica-rich layers?
- Unstable slopes?

## 3.5 MUDROCKS

Included under this heading are all sedimentary rocks formed by the consolidation and cementation of sediments which are predominantly clays or silts or clay-silt admixtures. The common rock types are:

- Claystone (predominantly clay sizes);
- Siltstone (predominantly silt sizes);
- Mudstone (clay-silt admixtures);
- Shale (any of the above, but fissile due to well-developed cleavage parallel to the bedding).

The sediments may have been deposited in either marine or fresh water conditions and usually have been derived from erosion of older rocks. In some cases they contain particles of volcanic origin. Possible cementing agents include calcite, silica, iron oxides and evaporite minerals such as gypsum, anhydrite and halite (common salt). Mudrocks occurring in or associated with coal-bearing sequences often contain abundant carbonaceous material and sulphide minerals, e.g. iron pyrite.

### 3.5.1 *Engineering properties of mudrocks*

Most mudrocks when fresh lie in the weak to very weak range as defined on Table 2.4. The very weak claystones grade into hard, overconsolidated clays. The strongest mudrocks lie in the medium strong and strong ranges and in most cases these owe their greater strength to cementation by calcite or silica.

Because of their relatively high clay contents the porosity and water absorption properties of mudrocks are much higher than those of most other rocks. As a result of this and the expansive nature of clays, all mudrocks swell and develop fine cracks on prolonged exposure to wetting and drying. The strongest siltstones, which contain appreciable amounts of calcite or silica as cement, can be exposed for up to a year before cracks are evident. At the other end of the scale, the weakest claystones and shales develop fine cracks as soon as they dry out (often only hours or days) and disintegrate with further cycles of wetting and drying. Some of these materials also swell noticeably on removal of overburden.

The mechanisms involved in deterioration of mudrocks on exposure have been described already in Sections 2.5.3 and 2.9.1.

Because of their instability when exposed, special care needs to be taken during preparation of foundations on mudrocks. Treatments range from shotcreting or slush concreting immediately after exposure and cleanup, to an initial cleanup followed by a final cleanup immediately before placement of concrete or fill.

Taylor and Spears (1981), Cripps and Taylor (1981) and Spink and Norbury (1993) provide useful information about the engineering properties of mudrocks occurring in the United Kingdom.

Mudrocks containing iron pyrite or other sulphide minerals can cause severe problems for dam projects due to rapid weathering of the sulphides to form sulphuric acid and metallic hydroxides and sulphates. The rapid weathering processes and some of their effects on dam projects are described in Section 2.9.4. Such effects may include:

- Damage to the fabric of concrete due to sulphate attack;
- Heave of foundations or of excavation sides causing damage to concrete slabs or walls (Penner et al., 1973; Hawkins and Pinches, 1987);
- Blockage and/or cementation of filter zones or drains;
- Seepage waters which are acidic and rich in iron or other heavy metals;
- Possible lowering of shear strength, of embankments formed by pyritic mudrocks.

### 3.5.2 *Bedding-surface faults in mudrocks*

Valley bulging (see Chapter 2, Section 2.5.4 and Figure 2.7) is a common feature in mudrock sequences, where the beds are near-horizontal. In such situations and in any other situation in which mudrock sequences have been disturbed by folding, tilting, or stress relief movements, thin seams of crushed rock develop within the mudrocks, usually at their boundaries with interbedded stiffer rocks (e.g. sandstones or limestones). These seams are developed due to interbed slip during the movements, in the same way as foliation shears develop in schistose rocks (Figures 2.8, 3.16 and 3.19). In mudrocks they are known as bedding-surface faults, bedding-surface shears or bedding-plane shears. They usually consist mainly of clay, are almost planar and have slickensided surfaces both within them and at their boundaries. Residual effective shear strengths of laboratory sized samples (i.e. excluding larger scale roughness effects) are commonly in the range 7° to 12°, with zero cohesion.

Although usually extending over wide areas, bedding surface faults may be only a few millimetres thick. Such defects are difficult to recover and recognize in diamond drill cores. This is particularly so when the defect is normal to the axis of the borehole, as even minor



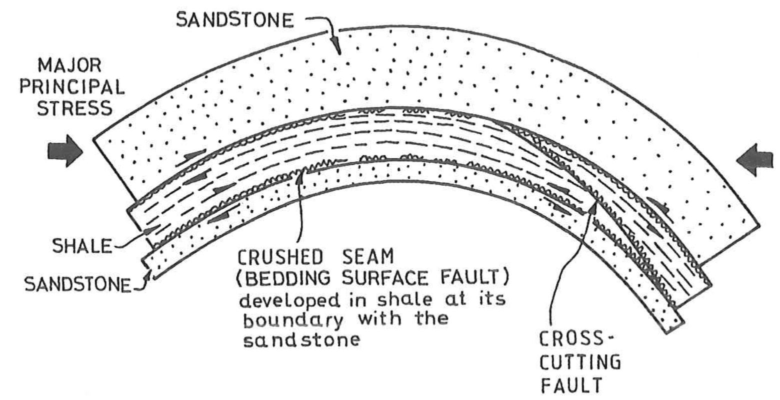



Figure 3.19. Usual way in which bedding surface faults are formed in mudrocks.

core rotation causes remoulding of the defect, which is then difficult to distinguish from remoulded mudrock at a "drilling break" in previously intact core (see Section 3.5.6).

The influence of bedding surface faults on the design of several embankment dams is described by Casinader (1982). Their effects on the design and construction of Sugarloaf and Thomson Dams are discussed in Chapter 2, Section 2.10.

Maddox et al. (1967) describe their effects at Meadowbanks Dam, a 43 m high massive buttress dam in Tasmania, founded on interbedded sandstones and mudstones. The dam was planned originally to be a concrete gravity structure but, during excavation of the foundation, continuous bedding-surface seams of clayey silt were found within the mudstones. Change to the buttress design and installation of prestressed cables were needed to achieve the required factor of safety against sliding.

### 3.5.3 *Slickensided joints or fissures*

Many mudrocks, particularly claystones, contain zones in which the rock contains an irregular network of curved, intersecting, slickensided joints. These joints are believed to have originated when the material was still a clay soil and to have developed by any of the following types of process:

- syneresis (Skempton and Northey, 1952; White, 1961);
- shrink and swell movements (Corte and Higashi, 1964);
- differential shear movements during consolidation;
- large lateral stresses (Aitchison, 1953; Terzaghi, 1961).

Due to their lack of continuity and their curved, irregular nature, these joints usually do not form continuous zones of very low shear strength. However, the shear strength of the jointed mass is appreciably lower than that of the intact mudrock. In adopting strength parameters for use in design, the strength of both the intact substance and the joints, and the spacing, orientation and continuity of the joints, need to be taken into account.

### 3.5.4 *Weathered products and profiles in mudrocks*

Weathering of mudrocks usually involves mechanical disintegration as described in Section 3.5.1 and the removal of cements such as calcite and silica. In the extremely weathered condition all mudrocks are clays or silts. Intermediate weathered conditions (e.g. slightly and distinctly) are often difficult to define in the weaker mudrocks, which when fresh are only a little stronger and more durable than hard clays. The Geological Society (1995) provides a suggested approach for site-specific mass-type classification of weathered profiles in these weaker mudrocks.

Weathered profiles in mudrocks are more uniform, gradational and generally not as deep as those in other rocks. Deere and Patton (1971) describe these types of weathered profiles in shale and point out that the lack of distinct boundaries within such profiles has led to contractual disputes over the depth to acceptable foundations.

### 3.5.5 *Stability of slopes underlain by mudrocks*

Slopes underlain by mudrocks commonly show evidence of past instability, even when the slope angles are small (e.g. 10° to 15°). This is not really surprising when we consider that all of the characteristics described above in Sections 3.5.1 to 3.5.4 tend to lower the strength of rock masses containing mudrocks.

Relatively shallow landsliding is common in the residual clay soils developed on the weaker mudrocks. Taylor and Cripps (1987) provide a comprehensive review of slope development and stability in weathered mudrocks and overconsolidated clays, mainly relating to examples in the United Kingdom.

Deere and Patton (1971) give a useful review of experience with unstable slopes on shales and on shales with interbedded sandstones. They point out that, in common with most other rocks, weathering of shales usually produces a low-permeability zone near the surface, underlain by jointed, less weathered shale, which is more permeable. Instability can arise when groundwater transmitted through this lower zone or along sandstone beds causes excessive pore pressures in the near-surface, more weathered shale. (See Section 3.6.4 and Figures 3.23 and 3.24).

The most common situations in which larger scale landsliding occurs, or is likely to occur, are where the bedding "daylights" on a valley slope. This occurred at both Sugarloaf and Thomson dams in Victoria, Australia, as described in Chapter 2, Section 2.10.

### 3.5.6 *Development of unusually high pore pressures*

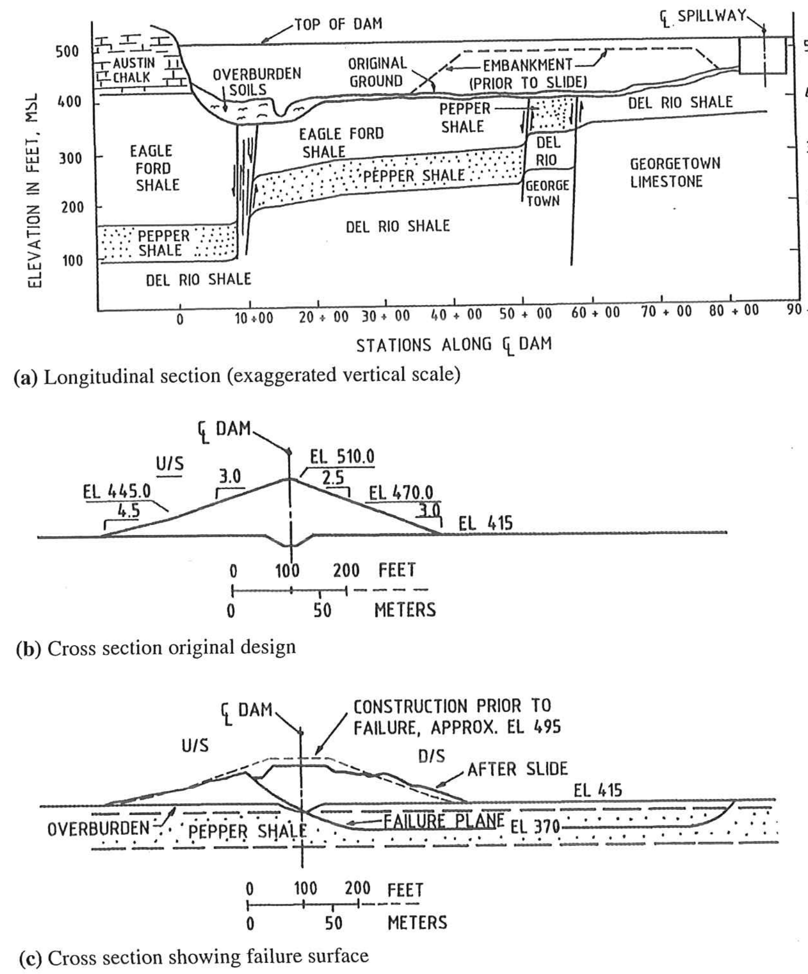
Stroman et al. (1984) and Beene (1967) describe a major slide which occurred during the construction of a 30 m high section of the 5.5 km long Waco Dam in Texas. The dam was located on near-horizontally bedded shales cut through by three steeply dipping normal faults, which displaced the shale units by up to 30 m and caused them to be locally folded (Figure 3.20a).

The originally designed embankment is shown in outline on Figure 3.20(b). The design was based on the assumption that the weakest foundation material was a localized 12 m thick layer with unconsolidated undrained strength of $\phi = 5°$, $c = 144$ kPa. No potential for pore pressure development was expected (because of the high degree of overconsolidation of the shales) and consequently no piezometers were installed in the original construction.

The failure occurred in a 290 m long section of embankment constructed between two of the normal faults. As can be seen on Figure 3.20c, the failure surface was mainly horizontal within the Pepper Shale about 15 m below the main foundation level. This failure surface broke out to the ground surface at an average distance of 235 m downstream from the dam axis.

Figure 3.21 shows the pore pressure distributions before the sliding and at the end of the reconstruction. Stroman et al. (1984) state that the unusually high pore pressures at the Pepper Shale/del Rio Shale contact were the cause of the slide. However they also report that direct shear tests on precut samples of Pepper Shale gave effective friction angles between 7° and 9°, with zero cohesion. They state that "This is the laboratory test






(a) Longitudinal section (exaggerated vertical scale)

(b) Cross section original design

(c) Cross section showing failure surface

Figure 3.20. Longitudinal and cross sections, Waco Dam (Stroman et al., 1984).

condition that can be related to a material that has been broken prior to construction and to the condition of the Pepper Shale after the slide".

Londe (1982) considers that the sliding at Waco Dam occurred by progressive shear failure (e.g. as described by Bjerrum, 1967, Terzaghi and Peck, 1967, Skempton and Hutchinson, 1969 and subsequently by Skempton and Coates, 1985, to explain the failure of Carsington Dam).

It seems equally possible to the present authors that very thin but continuous bedding surface faults with 7° to 9° residual strength may have existed in the Pepper Shale before the slide and contributed to the movement. Such bedding surface features require very small movements for their development and these could easily have occurred during the formation of the normal faults which bound the slide area. As described in Section 3.5.2, such minor bedding faults can be difficult to recover and recognize in drill cores.

### 3.5.7 *Suitability of mudrocks for use as construction materials*

Most mudrocks are not suitable for the production of materials for use in concrete, filters or pavements due to their generally low strengths and their slaking properties. However,

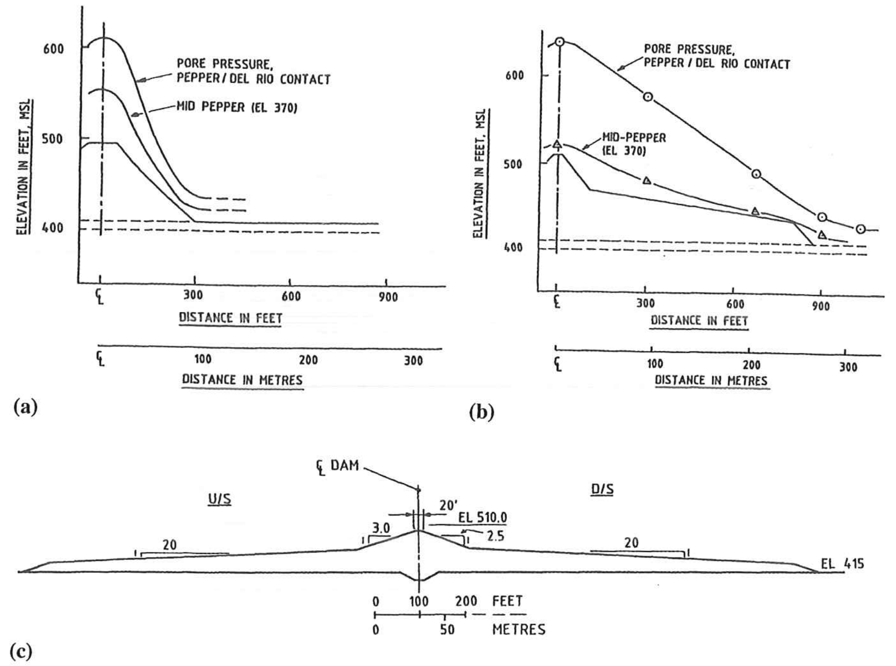



Figure 3.21. Waco Dam, pore pressure plots and cross section through embankment as rebuilt. (a) pore pressures prior to slide; (b) pore pressures after reconstruction; (c) embankment as reconstructed (Stroman et al., 1984).

near Sydney, a trial road pavement built using ripped and trafficked shale has performed satisfactorilly for 20 years (Won, 1985). The shale is weak to medium strong. Resampling and testing of the material showed that there had been little breakdown in service.

Siltstones in the medium strong to very strong range have been used successfully as rockfill for some years. Near the site of Sugarloaf Dam excavation of test pits into 100 year old tunnel spoil dumps comprising such materials, showed that slaking had not occurred in the rock below the near-surface two metres, i.e. in a near constant humidity environment.

To the knowledge of the authors no mudrocks have been used successfully as rip-rap. We would not expect it.

Random fills, earthfills and cores for embankment dams have been built successfully using mudrocks in various conditions ranging from fresh to extremely weathered (Vaughan 1994).

### 3.5.8 *Mudrocks – check list of questions*

- Slaking or disintegration on exposure?
- Swelling on exposure?
- Valley bulging?
- Soluble minerals in beds or veins?
- Presence of sulphide minerals?
- Slickensided fissures?
- Progressive shear failure?
- Bedding surface faults or shears?



Table 3.1. Common characteristics of sandstones, arkoses and greywackes.

| | | Minerals | |
|---|---|---|---|
| Rock name | Particle shapes, grading | Most grains | Common matrix/cements |
| Sandstone | Usually rounded, one-size grains and less than 15% matrix or cement | Quartz, fragments of older rocks | Silica, clay, iron oxides, calcite, gypsum |
| Arkose | Sub-angular, often well graded, little matrix | Quartz plus at least 25% feldspar; some mica | Clay, iron oxides, silica |
| Greywacke | Angular, well graded down to clay matrix which is usually >15% of volume | Felspar, quartz hornblende, micas, rock fragments, iron oxides | Clay, and same as grains |

- Unstable slopes (shallow, in weathered materials)?
- Unstable slopes (deep-seated, if bedding in folded rocks daylights)?
- Possibility of high pore pressures, in layered sequences?
- Suitability for rockfill, random fill, earthfill and haul roads?

## 3.6 SANDSTONES AND RELATED SEDIMENTARY ROCKS

The following are the main rock types considered under this heading. They will be referred to as the sandstone group.

- Sandstones;
- Arkoses;
- Greywackes;
- Siltstones;
- Conglomerates.

Table 3.1 sets out some common characteristics of sandstones, arkoses and greywackes. Siltstones and conglomerates have a similar range in composition to those of the other rocks. All except the siltstones occur usually in thick beds. The siltstones may be thickly or thinly bedded.

Also included under this heading are the lightly metamorphosed equivalents of the above, e.g. quartzites, metasiltstones and metaconglomerates. If the original depositional environment of the particular rocks at a site is known, then many generalisations can be made about their mineral contents, fabrics, bed-thicknesses and sedimentary structures and about rock types likely to be associated with them. Detailed discussion of these sedimentological aspects is beyond the scope of this book and only a few features of particular importance in dam engineering will be described. For more details readers are referred to Pettijohn (1957), Pettijohn et al. (1972), Selley (1982) and Walker (1984).

### 3.6.1 *Properties of the rock substances*

When fresh the sedimentary rocks range from extremely weak, non-durable to very strong and durable. The strengths and durabilities depend upon the strengths and durabilities of the grains and of the cements or matrices and these can vary widely depending upon the environment of deposition and subsequent histories of the rocks.

Quartz-sandstones (and conglomerates in which most grains are quartz) are often stronger and more durable than arkoses and their conglomerate equivalents, because of the superior strength and durability of quartz. However sandstones often have significant porosity (5% to 20%) and may also be slightly permeable.

Greywackes tend to be stronger than sandstones due to the angularity and grading of their particles.

Silica cement usually occurs in strong, durable rocks and at the other extreme rocks cemented by clay or gypsum are usually weak and non-durable.

If gypsum or anhydrite is proven or suspected as a cement in a sandstone forming all or part of the foundation of a dam, its significance needs to be assessed carefully and special testing may be required, as discussed in Section 3.8.

The metamorphic rocks when fresh are usually stronger and more dense and durable than the equivalent sedimentary rocks.

### 3.6.2 *Suitability for use as construction materials*

Rocks in the sandstone group which lie in the strong to extremely strong range have been used successfully as rockfill and rip-rap in many dams. They are widely used also as aggregates in concrete.

However, in a few cases concrete containing quartzite or strong sandstone as aggregate has suffered expansion and cracking due to alkali-silica reaction. The authors recommend that any silica rich rocks intended for use as concrete aggregate be tested for reactivity (see Section 3.2.6).

The quartzites are often extremely strong and this together with the high content of quartz (Moh hardness = 7) makes them highly abrasive. This can result in high quarrying and handling costs. Also if the quarry-run rock is not well graded it can be difficult to compact, as little breakdown occurs under the roller.

The weaker rocks tend to be more porous and usually lose significant strength on saturation. Mackenzie and McDonald (1981, 1985) describe the use of sandstones and siltstones, mainly medium strong when dry, as rockfill in the 80 m high Mangrove Creek Dam in New South Wales. Both rock types lost about 50% of their strength on saturation.

The rock was compacted with up to 5% water by volume. Higher water quantities caused the material to become unworkable. The fills produced were of high density and moduli but of generally low permeability. The latter was allowed for by the inclusion of drainage zones of basalt and high quality siltstone.

At Sugarloaf (Winneke) Dam thinly interbedded siltstone and sandstone were used for rockfill and random fill in the 85 m high concrete faced embankment (Melbourne and Metropolitan Board of Works, 1981; Regan, 1980).

The rockfill zone material was fresh or slightly weathered and strong to medium strong. Compacted with up to 15% water it produced a dense, free-draining fill.

The rock used in the random fill zone was slightly to highly (or distinctly) weathered and medium strong to weak. Compacted at about 10% moisture content it produced a dense fill which was not free-draining and contained up to 20% of silt and clay fines which were dispersive. It was therefore underlain everywhere by a blanket of rockfill and its outer surfaces were protected by a thin layer of the rockfill.

### 3.6.3 *Weathering products*

The main effect of chemical weathering on rocks of this group is weakening due to removal or decomposition of the cement or matrix. As a result the rocks with silica or iron oxide cements are the most resistant to weathering. The sandstones and conglomerates become progressively weaker until at the extremely weathered stage they are usually sands or gravels. Quartzites and quartz rich sandstones produce relatively clean quartz sands,



while arkoses and greywackes usually produce clayey or silty sands. Siltstones usually produce clayey silts or clays.

Some sandstones (usually weak, porous types) are locally strengthened in the weathered zone, by the deposition of limonite in their pores.

It can be difficult in some sandstones (e.g. those with calcite or limonite cements) to distinguish effects of weathering from effects of processes involved in the formation of the unweathered rocks. This can make it impossible to classify the rocks in the usual weathered condition terms, e.g. those in Table 2.3.

### 3.6.4 *Weathered profiles, and stability of slopes*

Weathered profiles in the weaker, more porous rocks commonly show gradational boundaries between rock in various weathered conditions. This is also the case in the stronger, more durable rocks when they are closely jointed.

Where large contrasts occur between the resistance to weathering of interbedded rocks, sharp but irregular sawtooth shaped boundaries can occur as shown in Figure 2.23.

Figure 3.22 shows the type of mechanically weathered profile often developed close to cliffs formed by near-horizontal sequences of thick sandstone beds underlain by or interbedded with siltstones or shales. Crushed seams (bedding surface faults) occur along the bed boundaries and steeply dipping joints have opened up in the sandstones. If these effects only occur close to the side of the valley it is likely that they result mainly from interbed movements due to stress relief – the shales/siltstones expanding further out of the

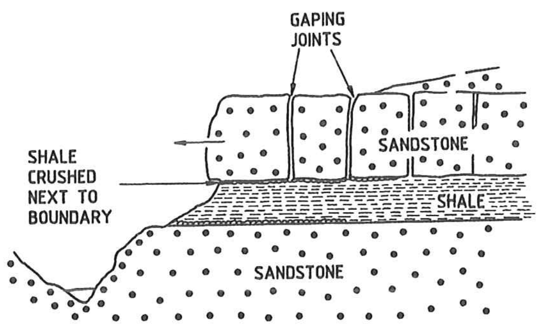



Figure 3.22. Commonly observed features close to cliffs formed by horizontally bedded sandstones with shale or siltstone interbeds. Based partly on Deere and Patton (1971).

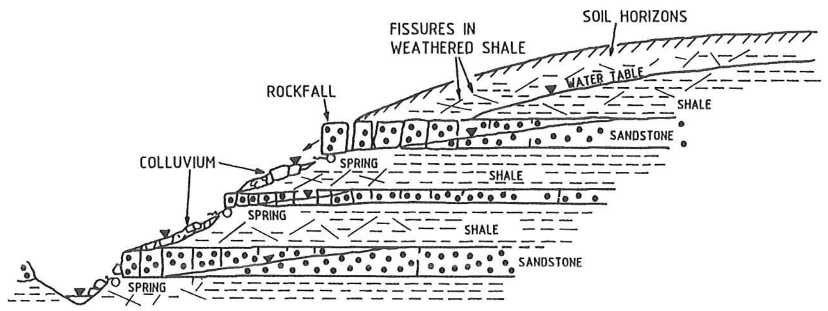



Figure 3.23. Collapse of outcropping sandstone beds due to removal of support of underlying shales (Based on Figure 13b of Deere and Patton, 1971).

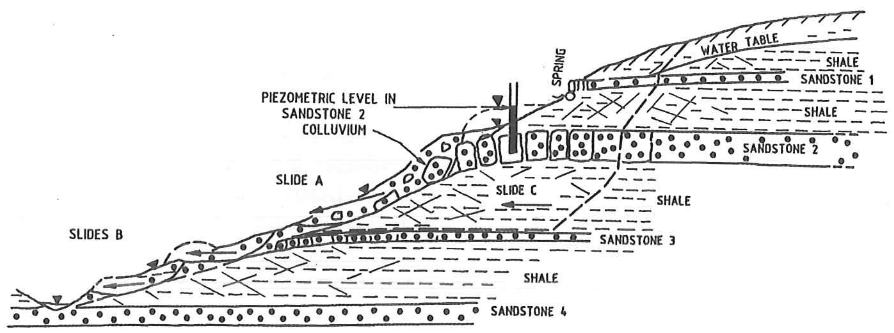



Figure 3.24. Landslides which can occur in a slope where sandstone beds are covered with colluvium (Based on Figure 13c of Deere and Patton, 1971).

slope than the sandstones. Joint-water pressures during extreme rainfall periods and earthquake forces may also contribute to the slight movements of the sandstone blocks.

Opening up of the joints in the sandstones causes the permeability of layers near the surface to be greatly increased.

Where whole hillsides are underlain entirely by rocks of the sandstone group, these rocks commonly form steep slopes or cliffs. Slope failures are rare and usually occur by rockfalls or toppling from the cliff portions.

More commonly, sandstones occur together with shales or siltstones as shown on Figures 3.23 and 3.24. In these situations weathering extends deeper into the mass and landsliding is more prevalent.

Figure 3.23, taken from Figure 13b of Deere and Patton (1971), shows a slope underlain by horizontally interbedded sandstones and shales. In high rainfall areas or during the wet season in other areas, the water table on Figure 3.23 will be high and of the form shown, due to the relatively free-draining nature of the sandstone beds. Springs occur at the bases of sandstone outcrops and the shale below and between the sandstone beds is either continually wet or alternately wet and dry. Under these conditions chemical weathering of both rock types occurs but is usually more pronounced in the shale. This rock is at least partly weathered to clay and contains clay-coated joints or fissures, often slickensided. The weathered shale either slumps or its bearing capacity is exceeded, allowing large movements and eventual collapse of the outermost sandstone block. Continuation of these processes leads to development of layers of scree and colluvium on the slope, and to "cambering" of the near-surface part of the sandstone bed as shown on Figure 3.24.

Deere and Patton also describe landsliding observed commonly on interbedded sandstone/shale slopes where sandstone beds have become covered by colluvium, as shown on Figure 3.24. The colluvium restricts drainage from the sandstone (Bed 2) which may become a semi-confined aquifer with piezometric surface as indicated. Pore pressures so developed cause sliding to occur, usually along the colluvium-weathered shale contact (Slides A and B on Figure 3.24).

Slide B is a double slide in which the toe of the first slide (No.1) has overloaded the metastable top of the slope below, causing Slide No.2 (Deere and Patton 1971).

Deere and Patton (1971) point out that deepseated slides, such as Slide C on Figure 3.24, can occur when a combination of unfavourable geological conditions exists. These could include bedding surface crushed seams along the shale-sandstone boundaries, high water pressures in sandstones Nos 2 and 3 and high water levels in the affected mass. Deere and Patton point out that after the first small movements, the permeability of the deeper part of the mass will be increased and softening and weathering of the near-surface shale will be accelerated.



In folded sequences of sandstones with interbeds of siltstones or shales, landslides are relatively common where dipping beds daylight on steep slopes. Slides are also common on dipslopes. Examples of these types of sliding involving sandstones are given in Chapter 2, Section 2.10.3.4 and 2.10.3.5.

### 3.6.5 *Sandstones and similar rocks – list of questions*

- Relatively high porosity, permeable?
- Gypsum or anhydrite present as cement?
- Quartzites: High quarrying and handling costs, difficult to compact?
- Rocks of medium or lower strength may not produce free-draining rockfill?
- Interbeds of shale or claystone?
- Bedding-surface faults at bed boundaries?
- Horizontal beds: Open joints and bedding surface crushed seams near surface due to stress relief?
- Horizontal beds with shale interbeds: Cambering and collapse due to removal of support by weathering shale?
- Landsliding in colluvium developed on weathering sandstone/shale slopes?

## 3.7 CARBONATE ROCKS

Carbonate rocks are defined here as those which contain significant amounts of the soluble minerals calcite, aragonite or dolomite in their substance fabrics. The most common are sedimentary carbonate rocks, marble (metamorphosed carbonate rock), and calc-silicate rocks formed by the metamorphism of impure carbonate rocks.

It is necessary to consider carbonate rocks in two categories, based on the dominant carbonate minerals present in each category.

*Geologically young carbonate rocks (Category Y). Category* Y carbonate rocks are usually of Tertiary, or younger, age. Most comprise loosely packed, weakly cemented shell fragments and are porous and weak to very weak. Rarely they can be well cemented and dense. The carbonate minerals are mostly aragonite and high-magnesian calcite (Friedman 1964, 1975; Molenaar and Venmans, 1993; Prothero and Schwab, 1996). These two minerals formed all original marine carbonate sediments and are forming in present day deposits. There is field and laboratory evidence (see Sections 3.7.1.3 and 3.7.7.2) that high-magnesian calcite is more susceptible to dissolution and cementation than aragonite and calcite. With time, exposure to fresh water, compaction and recrystallisation both aragonite and high-magnesian calcite eventually revert to calcite, and the rock becomes *Category O*.

*Geologically old carbonate rocks (Category O). Category* O carbonate rocks are generally of Mesozoic or older age and are usually dense, non porous and range from strong to extremely strong. They are formed by the minerals calcite or dolomite. They include marble, which comprises coarsely crystalline calcite and is usually dense, non porous and strong to very strong. Also included are calc-silicate rocks, which contain carbonate minerals together with silicate minerals often including olivine, diopside and garnet.

The carbonate rocks at the sites of most large dams are *Category O*. The exceptions mentioned in this book are Kopili Dam, Perdikas, Montejagne and May Dams, each of which failed to store water (Table 3.4).

Table 3.2 proposed by Dearman (1981) is a practical engineering classification for sedimentary carbonate rocks. Dearman (1981), Cruden and Hu (1988) and Bell (1981, 1983b and 1992) present data on physical properties of some carbonate rocks. Fookes and Higginbottom (1975) provide a table which classifies and names various types of

Table 3.2. Engineering classification of sedimentary carbonate rocks (Dearman, 1981).

| Predominant grain size (mm) | Percentage Carbonate 0–10 | 10–50 | 50–90 | 90–100 | |
|---|---|---|---|---|---|
| | | | LIMESTONE | | |
| > 2 | CONGLOMERATE | Calcareous Conglomerate | Gravelly Limestone | Calcirudite | LIMESTONE |
| 2 – 0.06 | SANDSTONE | Calcareous Sandstone | Sandy Limestone | Calcarenite | LIMESTONE |
| 0.06 – 0.002 | MUDSTONE: SILTSTONE | Calcareous Siltstone | Silty Limestone | Calcisiltite | LIMESTONE |
| < 0.002 | MUDSTONE: CLAYSTONE | Calcareous Claystone | Clayey Limestone | Calcilutite | LIMESTONE |
| | | MARLSTONE | MARLSTONE | | |

Note : non-carbonate constituents are rock fragments or quartz, micas, clay minerals.
: predominant grain size implies over 50%.

| PERCENTAGE CALCITE | 0–10 | 10–50 | 50–90 | 90–100 |
|---|---|---|---|---|
| | Dolomite | Calcitic Dolomite | Dolomitic Limestone | Limestone |
| PERCENTAGE DOLOMITE | 100–90 | 90–50 | 50–10 | 10–0 |

original pure carbonate sediments (detrital, chemical and biochemical) and their progressively strengthened counterparts. This table has useful descriptive terms and its accompanying discussion indicates some of the complexities involved in the strengthening of carbonates.

Calcrete (caliche) is a *Category Y* carbonate rock of highly variable strength found in many arid areas. The most common variety, pedogenic calcrete, occurs within a metre or so of the ground surface (Netterberg, 1969, 1971; James and Coquette, 1984; Meyer, 1997).

### 3.7.1 *Effects of solution*

Solution is one of the processes of chemical weathering which affects all rock types to some extent. It is more severe and causes cavities in carbonate rocks because calcite and dolomite are relatively soluble in acidic waters e.g. carbonic (dissolved carbon dioxide), organic (from vegetation) or sulphuric (from volcanic activity or oxidation of sulphides). They are more soluble in saline water than in pure water and their solubility increases with desceasing water temperatures. Also, their rates of solution are high (James and Kirkpatrick, 1980; James, 1981).

The term karst is used to describe terrain underlain by cavernous rocks (usually carbonates or evaporites) and also to describe the cavernous rocks themselves. The factors involved in the formation of karst are the same as those which contribute to the development of weathered profiles described in Chapter 2, Section 2.6.4. Detailed discussion of karst development, structure and hydrogeology can be found in Sweeting (1972, 1981), Milanovic (1981), Bonacci (1987), Ford and Williams (1989), Beck (1993, 1995) and De Bruyn and Bell (2001).

The effects of solution as seen in the most common types of carbonate rock masses are discussed below.



Figure 3.25. Karst landscape in New Zealand. The outcrops of limestone are up to 1.5 m high and separated by soil-covered holes and slots up to 1 m across.

#### 3.7.1.1 *Rock masses composed of dense, fine grained rock substances comprising more than 90% of carbonate (usually Category O)*

When fresh and intact these rocks usually have very low porosities and their substance permeabilities are effectively zero. Groundwater flow is confined to joints or other defects and these are widened by solution to form slots, shafts, tunnels and cavities of all shapes and sizes. Important features are:

- The upper surface of rock, both in outcrops and below ground, usually consists of pinnacles of fresh rock separated by deep slots or cavities, which may be empty or soil-filled (Figures 3.25 and 3.26);
- No weathered rock substance is present, either at the ground surface or at depth. This is because dissolution of the relatively pure limestone substance leaves less than 10% of insolubles, which form residual soils at or near the ground surface and infill some cavities below (Figure 3.27). The residual soils are commonly fissured and rich in clay and iron oxides;
- Dam construction experience has shown that cavities capable of accepting or transmitting large inflows of water can be met at any depth down to 300 m below ground surface (see Section 3.7.2.1);
- Sinkholes, usually shaft-like cavities, are often exposed at the ground surface and the formation of new sinkholes can be initiated by natural or man-made activities.

Figure 3.27 represents the uppermost 10 m or so of a karst profile beneath a small area, showing relatively small cavities and solution-widened defects, which would be connected to larger cavities at greater depths. It also illustrates some of the mechanisms by which sinkholes have been formed.

Feature (a) is a sinkhole formed directly by the collapse of the roof of a cavity.

Feature (b) can also be referred to as a sinkhole. It is a gentle, soil-covered depression, roughly circular in plan view. Exploration shows it to be underlain by residual

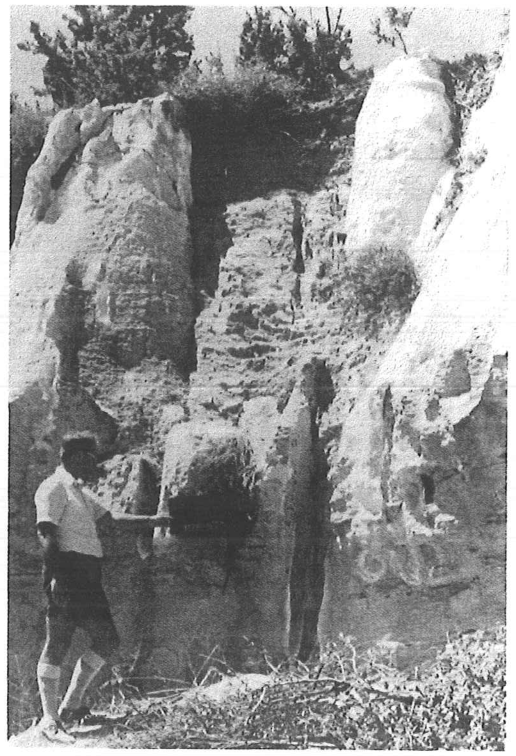


Figure 3.26. Close up view of limestone pinnacles.

soil with a concave upwards layered structure. It can be inferred that this sinkhole has developed because the residual soil has migrated slowly into the cavities in the limestone below.

Feature (c) is a large flat area showing residual soil at the surface. It is underlain at depth by cavities developed along joints and a fault zone. Such flat areas may show no obvious evidence of settlement or disturbance, but there is a significant risk of a deep, steep-sided sinkhole forming (with little or no warning) at the ground surface. This could be initiated by migration of soil into the cavities below, causing a large void to develop near the base of the residual soil. The sinkhole could then form by collapse of the overlying soil into the void. A similar dangerous sinkhole could also develop within the gently sloping sinkhole (b).

The mechanisms of sinkhole formation and their significance in dam engineering are discussed further in Section 3.7.3.

#### 3.7.1.2 *Rock masses composed of dense fine grained rock substance containing 10% to 90% of carbonate (usually Category O)*

In these rocks the pattern of solution cavities is similar to that described in Section 3.7.1.1 and seen in Figure 3.27, but usually the rock substance next to some of the cavities is weathered. Experience and logic show that in general the lower the percentage of carbonate in the fresh rock the higher the proportion of weathered rock formed compared to cavities and the higher the ratio of infilled cavities to open cavities. The weathered rock is usually much weaker and less dense than the fresh rock. However, the proportion and properties of weathered rock formed depend also on the percentage of insoluble particles present in the fresh rock and the degree to which these particles are bound by non-soluble



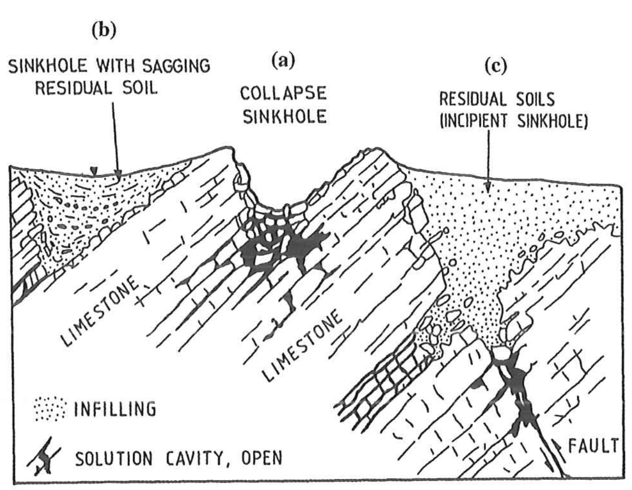



Figure 3.27. A typical weathering profile in dense, relatively pure carbonate rock (Based on Deere and Patton, 1971).

cements. These effects are illustrated by strong dolomitic siltstones which occur widely in South Australia. Many weathered exposures of these rocks appear to have lost all of their dolomitic cement, but are still weak or medium strong rock, due to some non-carbonate cement. A few beds within such a sequence at and near Little Para Dam appear to lack this non-soluble cement and these beds form very low density, very weak rock, grading to clayey silt and small cavities (see Figure 3.33 in Fell et al., 1992).

#### 3.7.1.3 *Rock masses composed of porous, low density carbonate rock substance (usually Category Y)*

Most of these rocks are weak to very weak calcarenites. In many cases the carbonate grains are recognizable shell fragments. The rocks are usually in horizontal or gently dipping beds and are relatively free from joints or other defects of tectonic origin.

These rocks have sufficient substance permeability to allow surface waters to enter and migrate downwards, causing both solution and redeposition effects. The 8 m high quarry face in Figure 3.28 shows a typical profile. The very weak calcarenite here is a windblown deposit of Pleistocene Age. It is off-white in colour and overlain by yellow-brown quartz which appears dark grey in the photograph. The profile in the calcarenite is characterized by the following:

- Vertical pipes or tubes; each is surrounded by an annular zone of calcarenite which is much more dense and strong than elsewhere in the mass. The tubes have been infilled with the quartz sand, so show up well where exposed on the face;
- Vertical pinnacles of calcarenite which are much more dense and strong than elsewhere in the mass.

The tubes have been formed by solution of the calcarenite. Strengthening of their rocky surrounds and of the pinnacles appears to have been caused by redeposition of calcite derived from that solution process and possibly also from solution of calcarenite which previously existed above the present ground surface.

The porous, low-density carbonates rarely contain large cavities like those in dense, jointed carbonates. However Twidale and Bourne (2000) describe many shallow sinkholes up to 10 m across, and several much larger, on wind-deposited calcarenite in South

Australia. They present evidence which suggests that these have developed due to solution along localized calcarenite zones rendered more permeable by small movements along faults in the bedrock at a depth of 100 m or more.

Chalk is very low strength, low-density calcilutite (Table 3.2, and Table 13.8 in Bell (1983b). It has extremely low substance permeability, but can be permeable where joints are present. Bell (1983b) and Goodman (1993) point out that large cavities are rare in chalk, but solution pipes and sinkholes can occur, near its contact with colluvium or other rocks.

### 3.7.2 *Watertightness of dam foundations*

Many dams have been built successfully on sites underlain by *Category O* carbonate rocks, despite the fact that solution cavities have been present at most sites and beneath at least part of their storage areas.

Selection of dam foundation levels and treatments can be difficult, due to the highly irregular nature of the uppermost surface of the fresh rock, as shown by Figures 3.25 to 3.28. This applies particularly to embankment dams, because of their large foundation areas and the need to provide stable, non-erodible surfaces for placement of embankment materials.

Treatments to fill cavities and prevent excessive leakage have included cement grouting, concrete curtain walls and selective mining and backfilling with concrete. The presence of clay infilling in cavities presents a problem when cement grouting is proposed. Clay usually prevents grout penetration and is not readily removed by flushing between boreholes. However, if left in place, it may be flushed out later when the dam is filled. High pressure cement grouting designed to cause hydraulic fracturing of clayey infills has been shown by Zhang and Huo (1982) to give significant improvement in their resistance to leakage and piping.

At the 35 m high Bjelke-Peterson earth and rockfill dam in Queensland about 250 m of the left bank was formed by a landslide deposit underlain by limestone. In this section the cutoff trench was excavated down to the water table, 15–20 m below the original surface and 5–15 m below the irregular upper surface of the limestone (See Chapter 8, Figure 8.8). At the trench base the limestone was mainly fresh, dense and very strong, but contained open and infilled solution cavities within and next to sheared and crushed zones (Eadie,

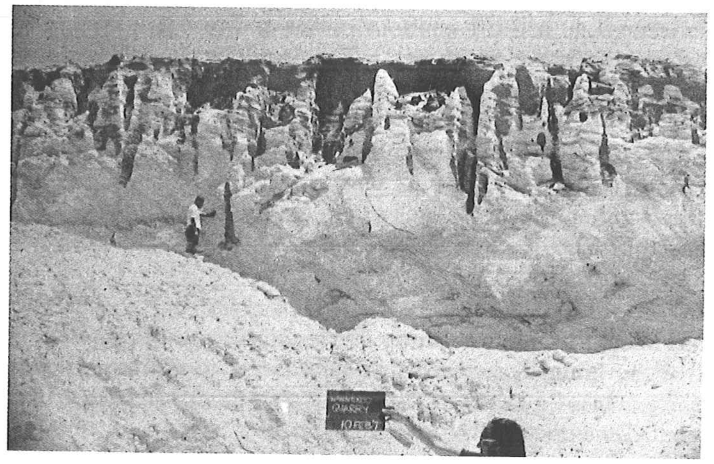


Figure 3.28. Weathering profile on very weak, porous calcarenite near Perth, Western Australia. Note the extremely irregular nature of the rock surface.



1986; McMahon, 1986). One of these zones was locally deepened by up to 8.5 m and backfilled with concrete. Throughout the limestone sections the base of the trench was covered by a reinforced concrete slab anchored into the rock. Cavities exposed on the downstream face of the trench were sealed with concrete. A single row grout curtain below this grout cap used 32 m deep vertical holes at 0.5 m spacing. Each 8 m stage, before testing and grouting, was flushed by high pressure air and water until the wash water was substantially clear. McMahon (1986) believes that this washing was unsuccessful. Relatively high grout pressures were used (3 times the reservoir head), aimed at achieving hydraulic fracturing of the clayey infill materials, like that reported by Zhang and Hao (1982).

The general foundation levels for the rockfill shoulders were reached by scrapers. In the limestone areas the exposed rock was essentially strong mounds and pinnacles separated by clay-filled depressions. These were cleaned out to the depth at which the clay filling was less than 1 m wide and backfilled with high quality rockfill.

At sites where cavities are numerous and/or large and partly or wholly filled with clayey soils, cement grouting alone is not usually relied on to form the cutoff. Other methods have included

- Cleaning out of some individual caves and backfilling with concrete;
- Walls formed by mining out slots of cavernous rock and backfilling with concrete;
- Diaphragm walls comprising overlapping boreholes backfilled with concrete;
- Closely spaced drilled holes, washed out with compressed air and water and backfilled with high-slump mortar, poured in and needle vibrated.

The last three methods were used, together with cement grouting, at the 85 m high, 1050 m long concrete face rockfill Khao Laem dam in Western Thailand. At the dam they were used to construct the upper 20–60 m of a curtain which extended up to 200 m (but generally less than 100 m) below the plinth. In one section, high pressure flushing with air and water was used to flush out sandy silt (weathered calcareous sandstone) from cavities and open joints before grouting.

Another curtain, 3.5 km long and up to 200 m deep, was constructed beneath the right abutment ridge, which was formed entirely by cavernous limestone, with its water table below the proposed storage level. All four methods were used in parts of this curtain, together with cement grouting. Lek et al. (1982) and Somkuan and Coles (1985) describe details of the methods.

#### 3.7.2.1 *Dams which have experienced significant leakage problems*

Table 3.3 lists dams which have recorded very high leakages, and Table 3.4 lists others which have never stored water because the leakage rates exceeded the inflow. Two recently completed dams which have recorded very high leakages are discussed below.

*Attaturk Dam.* Riemer et al. (1997) describe cavities met during the planning and construction of this 179 m high, 1800 m long rockfill dam in Turkey, completed in 1990. The site is formed by a folded sequence of *Category O* limestones including marly, cherty and bituminous units. The rock substances are moderately strong to weak, dense and impermeable. The rock mass is cut through by many steeply dipping faults and joints. Solution along these has resulted in a network of chimneys and near-horizontal tunnels. Despite the use of many adits and drill holes, the design stage studies failed to indicate the magnitude of this cavity network and of the grouting program required.

The main 3-row grout curtain is 5.5 km long and extends generally 175 m below the foundation with a local extension down to 300 m. Even at these depths the curtain is described by Riemer et al. (1997) as "suspended". An elaborate monitoring program involving 3.6 km of adits and about 300 piezometers has indicated seepage discharge rates

Table 3.3. Dams at which very high leakage rates were recorded (Based on Erguvanli, 1979; Riemer et al., 1997; Salembier et al., 1998).

| Dam | Country | Rock | Maximum recorded leakage ($m^3/s$) |
|---|---|---|---|
| Hales Bar | USA | Carboniferous limestone and shale | 54 |
| Keban | Turkey | Palaeozoic limestone | 26 |
| Lar | Iran | Mesozoic limestone, volcanics, lake deposits | 16 |
| Attaturk | Turkey | Palaeozoic limestones, folded and faulted | 14 |
| Great Falls | USA | Limestone | 13 |
| Camarasa | Spain | Jurassic dolomitic limestone | 12 |
| Dokan | Iraq | Cretaceous dolomitic limestone | 4 to 5 |
| Fodda | Morroco | Jurassic limestone | 3 to 5 |

Table 3.4. Dams which failed to store water (Erguvanli 1979).

| Dam | Country | Rock |
|---|---|---|
| Civitella Liciana | Italy | Cretaceous limestone |
| Cuber | Spain | – |
| Kopili | India | Eocene limestone* |
| May | Turkey | Mesozoic and Tertiary limestone * |
| Montejagne | Spain | Mesozoic and Tertiary limestone* |
| Perdikas | Greece | Miocene limestone* |
| Villette Berra | Italy | – |

* Geologically young carbonate rock (*Category Y*) present.

are governed mainly by reservoir level. The (1997) total discharge is estimated as 11–14 $m^3$/sec, with reservoir levels ranging from 6–8 m below FSL. These losses are tolerable, as the average flow of the impounded river is 850 $m^3$/sec. The monitoring also shows some high piezometric levels downstream from the curtain and some decrease in head upstream of it. Riemer et al. (1997) point out that these results "could hint at" ongoing erosion, either of grout or clayey infilling and they foreshadow the possible need for further grouting and/or drainage works.

*Lar Dam*. This is an earthfill embankment dam, 110 m high and 1100 m long, across the valley of the Lar River in Iran. Djalaly (1988) and Salembier et al. (1998) summarise the site geology and describe difficulties met since completion of the dam construction in 1980. Uromeihy (2000) provides more detail on the geology. The site is at the foot of an active volcano. Bedrock in the area is complexly folded, faulted and karstic limestones and mudrocks of Mesozoic Age. The dam and 15 km long reservoir area are located partly on these rocks and partly on near-horizontal materials of Quaternary Age. These include lava, ash, alluvial and lake deposits (mainly sandy and silty). They fill an old valley, which is around 200 m deep at the dam site and up to 600 m deep in the reservoir area. Before construction there were several large sinkholes (up to 25 m across and 30 m deep) in the alluvial/lake deposits in the reservoir area. The water table in the dam area was more than 200 m below the river bed and there was a karstic spring 10 km downstream from the dam, flowing at about 0.5 $m^3$/sec.

Uromeihy (2000) notes that outgassing $CO_2$ and $H_2S$ from hot springs flanking the volcano would increase the acidity of the streams draining into the reservoir but provides no data on the chemistry of the reservoir water.

After impoundment started in 1981, large new sinkholes appeared in alluvium. These were exposed when the lake was drawn down and many depressions in the lake floor were located by sonar survey. Uromeihy (2000) showed that 8 large sinkholes within 6 km upstream from the dam lie close to the trace of a known fault. The water table near the dam rose by 80–100 m. New springs appeared downstream and flow from the original spring increased to 9.9 $m^3$/sec. The total flow rate from all springs indicated that leakage from the reservoir was about 16 $m^3$/sec. This was more than the average inflow at the site and the highest reservoir level reached was 23 m below Full Supply Level. After studies and analyses of the leakage, site exploration, grouting and other remedial works were commenced in 1983. These involved 55,000 m of drilling and 100,000 tonnes of injected materials. The largest cavern located and treated was 200 m below the river bed, and was 27 m high and 68 m wide. Smaller cavities were located down to 430 m below the river bed (Djalaly, 1988). The treatment works did not lower the leakage rate and were stopped in 1990.



In 1991 site studies started again, but with a new emphasis, as indicated in the following words by Salambier et al. (1998) "*The key to understanding the leakage conditions and consequently proposing the most appropriate measures was geological modelling based on extensive investigation and comprehension of the geological history of the site*".

The scope of the new studies is summarized by Salambier et al. (1998). They included geological surface mapping, interpretation of air photographs and satellite imagery, an extensive geophysical program and drilling 10 holes totalling 3,750 m. The deepest drill hole was 650 m.

This work resulted in improved geological and hydraulic models from which Salambier et al. (1998) showed that the extent of permeable ground was much greater than previously assumed and that there was a serious problem with potential sinkhole formation (see Section 3.7.3).

They also predicted the extent of ground requiring treatment to solve the leakage and associated problems and were able to delineate this in plan view and on sections. In the bedrock beneath the dam and its right abutment the treatment area continued to 600 m below river bed level as shown on Figure 3.29. Under the storage area the treatment area was shallower, but extended more than 5 km upstream from the dam.

Salambier et al. (1998) concluded that "successful plugging and grouting treatment cannot be guaranteed" and that the cost could not be estimated "within a reasonable range of certainty". They recommended that, as an alternative, blanketing of the ground surface be considered – by shotcrete and poured concrete on the limestone and by geomembrane, over the alluvial/lake deposits. Clay was considered feasible, but too scarce near the site. Reimer (personal communication) advises that as at mid-2003 no work has been done.

### 3.7.3 *Potential for sinkholes to develop beneath a dam, reservoir or associated works*

Sinkholes which develop naturally (Figure 3.27) are present in most areas underlain by carbonate rocks. In many such areas collapse of the ground surface to form a new sinkhole is a relatively common occurrence. Some of these collapses may happen naturally but more often they are induced or, accelerated by man's activities, as seen at Lar Dam. Possible mechanisms for sinkhole formation include the following:

1. *Dewatering*. This can cause loss of buoyant support of the rock or soil forming the roof of a cavity or, by steepening the hydraulic gradient, cause erosion and collapse of overlying soils into the cavity. It may also cause collapse due to drying out, causing shrinkage and ravelling failure of overlying soils.
2. *Inundation*. This can cause previously dry soil to lose apparent cohesion and collapse or erode into a cavity or can steepen the gradient in previously wet soils, causing them to erode into a cavity or into joints widened by solution.
3. *Vibrations* from machinery or blasting.

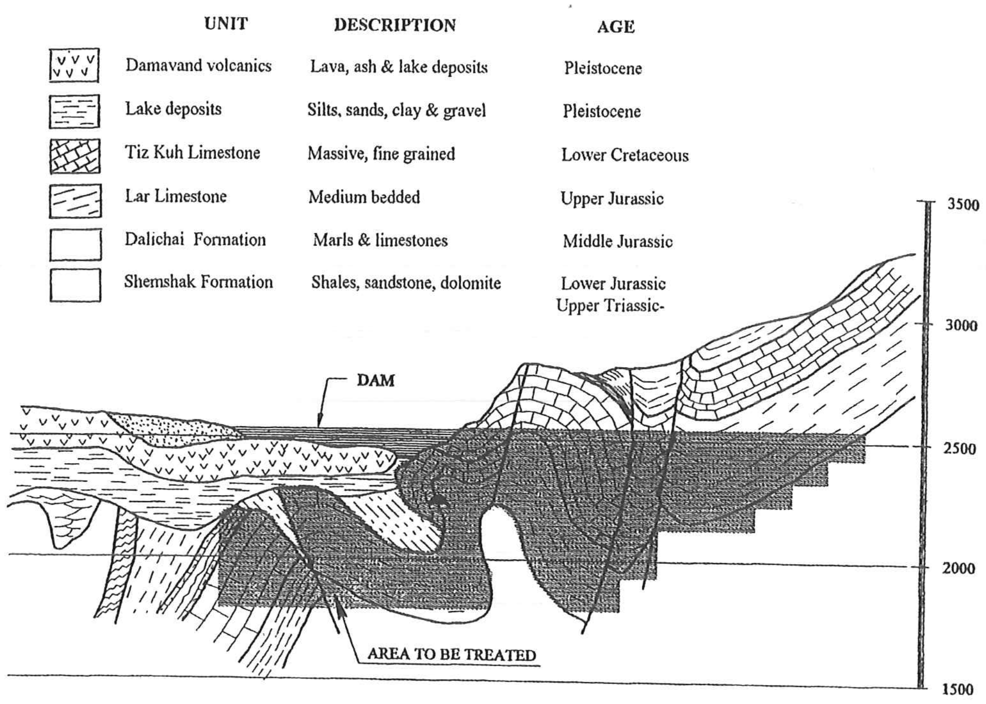



Figure 3.29. Lar Dam, cross section showing the predicted extent of ground treatment needed at the dam site (Salambier et al., 1998).

Examples of most of the above have been described by Sowers (1975), Brink (1979), Newton and Tanner (1987) and Wilson and Beck (1988).

Chen et al. (1995) have suggested that a rapidly rising water table in cavernous rock could cause a "water hammer effect" – pressure waves which might accelerate or induce collapse of overlying soils into cavities. First and subsequent fillings of a reservoir could provide these rapid rises in a water table.

It is clear that for all sites in karst areas there will be some risk of ground surface collapse affecting any part of the project works, including borrow areas and haul roads as well as the dam embankment and associated works. The worst imaginable event of this type would be a major sinkhole forming beneath a dam embankment during reservoir operation.

The possibility of this situation arising at Lar Dam has been accepted by Salambier et al. (1998). The dam has been extensively monitored, with instruments including standpipe, hydraulic and pneumatic piezometers. From a flow net for the dam and its foundation (Figure 3.30) they point out that:

- Vertical gradients exist in the upstream part of the core, in the upstream transition zone and in the alluvial foundation, and
- Piping of any of these materials is possible if the reservoir is operated at higher levels.

They conclude that "...development of a large sinkhole cannot be excluded. If this were to happen, complete failure of the embankment could occur".

Uromeihy (2000) warns that there is a risk of sinkhole development also beneath the downstream shoulder.



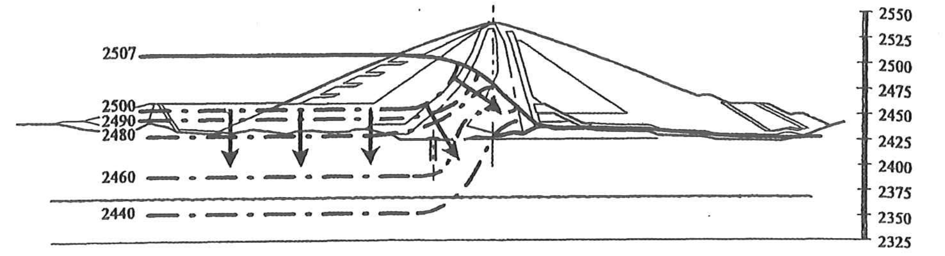



Figure 3.30. Lar Dam, flow net in the embankment and foundation (Salambier et al., 1998).

### 3.7.4 *Potential for continuing dissolution of jointed carbonate rock in dam foundations*

The following sections relating to dissolution processes and effects rely heavily on theoretical and experimental material described by James and Lupton (1978, 1985), James (1981) and James and Kirkpatrick (1980).

Most of the material in those papers is included and expanded upon in James (1992), a comprehensive book covering a wide range of aspects ranging from the fundamentals of solubility to the engineering behaviour of all soluble rocks, i.e. carbonates and evaporites. Also included are useful case histories of dams and other structures in which solution effects were investigated.

James and Lupton (1978) discuss the principles governing the dissolution of minerals, develop mathematical models which predict how the minerals gypsum and anhydrite dissolve in the ground and describe laboratory tests which confirm the validity of the predictions. Water flow through both jointed rock and porous granular material are considered. They show that the rate at which the surface of a mineral or rock retreats depends primarily upon two properties of the mineral, namely:

- the solubility (c) of the mineral, which is the amount which can be dissolved in a given quantity of solvent, at equilibrium, and
- the rate of solution of the mineral, which is the speed at which it reaches the equilibrium concentration.

The solution rate constant (K) is further dependent on the flow velocity and temperature of the solvent and concentrations of other dissolved salts.

James and Lupton (1978) point out that for water flowing through a joint in an effectively impervious but soluble rock substance, widening of the joint by solution of the rock walls will occur only until the water becomes saturated with the soluble material. Thus a solution zone (Figure 3.31a) is initiated near the fresh water interface and migrates progressively downstream. For water flow through soluble rock substance with intergranular (fabric) permeability a comparable solution zone is formed and migrates downstream (Figure 3.31b).

James and Kirkpatrick (1980) use the methods and models of James and Lupton (1978) to derive conclusions on foundation treatments needed to prevent dangerous progressive dissolution when dams are built on rocks containing gypsum, anhydrite, halite and limestone. They use the following solution rate equations to predict the ways in which the materials dissolve:

For gypsum, halite and limestone

$$\frac{dM}{dt} = KA(c_s - c) \tag{3.1}$$

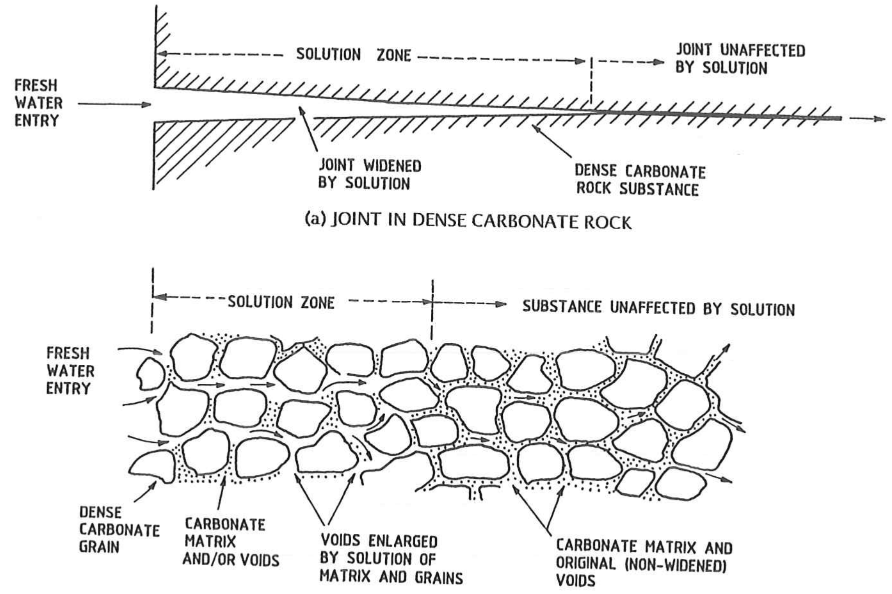



Figure 3.31. Development of solution zones in (a) a joint and (b) soluble rock substance or aggregate of carbonate particles (Based on James and Lupton 1978).

For anhydrite

$$\frac{dM}{dt} = KA(c_s - c)^2 \qquad (3.2)$$

where M is the mass dissolved in time t, A is the area exposed to solution, $c_s$ is the solubility of the material, c is the concentration of material in solution at time t and K is the solution rate constant.

The limestone used in their laboratory tests was Portland stone, a dense, old rock (Jurassic Age), in which the carbonate mineral is calcite. For this rock they showed that K values rose sharply at flow rates close to the onset of turbulence. They note that these "transitional" velocities (around 0.75–0.8 m/s) would be reached by water flow in joints with apertures of about 2.5 mm, with an hydraulic gradient of 0.2.

James and Kirkpatrick (1980) use mathematical modelling to provide predictions of the way joints enlarge as water flows through them. Their Figure 3, reproduced here as Figure 3.32, shows their predictions for the enlargement by pure water of limestone joints with initial apertures ranging from 0.5 mm to 2 mm.

It can be seen that the solution occurs essentially by downstream migration of the solution zone, with no enlargement further downstream. After a period of 100 years, the solution zone of the 0.5 mm open joint has migrated only 13 m from the inlet face. The 0.7 mm joint shows a similar type of behaviour, but the enlarged portion migrates 30 m in only 40 years. During very short periods the larger aperture joints show long tapered enlargements indicating that in real dam situations (i.e. seepage paths often much less than 100 m) seepage flows would accelerate.

James and Kirkpatrick (1980) conclude that the smallest aperture joint which will cause dangerous progressive solution in limestone is 0.5 mm for pure water or 0.4 mm for water



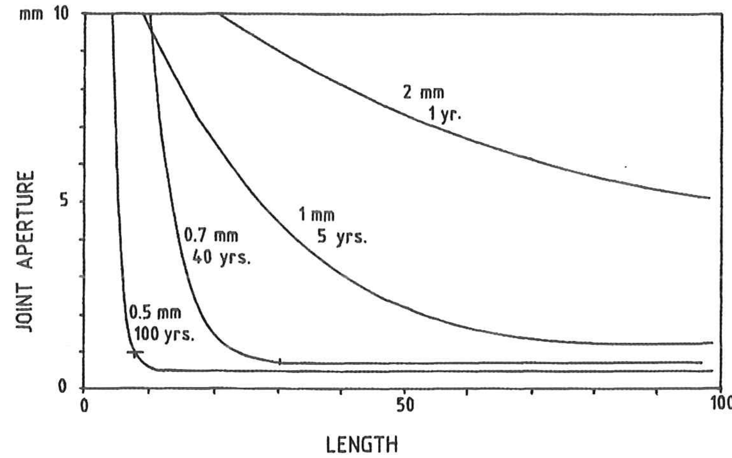



Figure 3.32. Enlargement of joints in dense *Category O* limestone (Portland stone) by pure flowing water (James and Kirkpatrick, 1980, reproduced by permission of the Geological Society).

Table 3.5. Solution of jointed rock (from Table 3 of James and Kirkpatrick, 1980).

| | Largest joint aperture (mm) | | |
|---|---|---|---|
| | For stable inlet face retreat | For rate of retreat of 0.1 m/year | Suggested preventive measures |
| Gypsum | 0.2 | 0.3 | Grouting |
| Anhydrite | 0.1 | 0.2 | Cut-off – e.g. cement-bentonite |
| Halite | 0.05 | 0.05 | Cut-off – e.g. cement-bentonite |
| Limestone (Category O) | 0.5 | 1.5 | Grouting |

Note: Values are for pure water; at joint spacing of one per metre and an hydraulic gradient of 0.2.

containing 300 mg/l of dissolved carbon dioxide. They further conclude that if all large cavities are backfilled with grout or concrete, then cement grouting (which can fill joints down to about 0.2 mm aperture) should be adequate to prevent progressive solution of limestone foundations. They point out the need for care in the conducting of Lugeon permeability tests and in estimating the apertures of joints from the results of the tests.

Table 3.5 summarises the conclusions of James and Kirkpatrick for preventing progressive solution along joints in all four soluble rock types.

### 3.7.5 *Potential for continuing dissolution of aggregates of carbonate rock particles, and of permeable carbonate substances (Category O carbonate, in each case)*

James and Kirkpatrick (1980) also discuss solution rates in "particulate deposits". It is assumed that such deposits might comprise either aggregates of rock fragments (as in some zones of sheared or crushed rock, or in filter zones) or substances rendered permeable by intergranular voids (Figure 3.31b). Using the theoretical models of James and Lupton (1978), James and Kirkpatrick show that the length of the solution zone depends mainly on the solution rate constant and the solubility and that other factors such as the

Table 3.6. Solution of particulate mineral deposits (James and Kirkpatrick, 1980).

| Mineral | Limiting seepage velocity (m/s) | Length of solution zone (m) |
|---|---|---|
| Gypsum | $1.4 \times 10^{-6}$ | 0.04 |
| Anhydrite | $1.6 \times 10^{-6}$ | 0.09 |
| Halite | $6.0 \times 10^{-9}$ | 0.002 |
| Limestone (Category O) | $3.0 \times 10^{-4}$ | 2.8 |

Note: Rate of movement of solution zone is 0.1 m/year. Mineral particles diameter 50 mm; pure water.

shape and sizes of the particles and the proportion of soluble minerals present, are of secondary importance. They show further that the downstream migration rate of the solution zone is governed largely by the seepage velocity and by the solubility, the proportion of soluble mineral present being less critical.

Table 6 of James and Kirkpatrick (1980), reproduced here as Table 3.6, shows the calculated seepage velocities which would be required to cause 0.1 m/year downstream migration of solution zones in a range of such "particulate mineral deposits". Based on their opinion that the required seepage rate for "particulate" limestone ($3.0 \times 10^{-4}$ m/s) is so high that it would be "rarely tolerated in dam foundations" they conclude that control of seepage by a good grouting program or other means should prevent dangerous progressive solution of "particulate" forms of dense carbonate rock.

James (1981) determined the solution parameters for 10 carbonate rocks of different compositions – 8 limestones, 1 calcitic dolomite and 1 probable chalk. They ranged in age from Carboniferous to Cretaceous. Using finely crushed samples and pure water, he found their solubilities to be virtually the same as that of "pure calcium carbonate" (presumably calcite) and that differences in their solution rates were too small to be significant in engineering design.

James (1981) showed for one sample of limestone, that small amounts of carbon dioxide dissolved in the water lowered the solution rate by about a factor of 10, but caused large increases in the solubility. Under "Engineering Considerations" he confirmed the conclusions and recommendations of James and Kirkpatrick (1980), but recommended that the following were also needed when assessing the potential for progressive solution of carbonate rock in a dam foundation:

- Chemical analyses of the appropriate reservoir, river or groundwaters, and
- Laboratory tests to determine the solubility of the carbonate rock in these waters.

### 3.7.6 *Discussion – potential for continuing dissolution of carbonate rocks in foundations*

#### 3.7.6.1 *Category O carbonate rocks*

A conclusion which might be drawn from the work of James and Kirkpatrick (1980) and James (1981) as discussed in Sections 3.7.4 and 3.7.5, is that dangerous ongoing solution of old, dense carbonate rocks in a dam foundation can always be avoided by a thoroughly planned and executed grouting program using Portland cement. However, early in their paper, under "Modelling of ground conditions", James and Kirkpatrick include the following warnings against such a general conclusion.

*"The conditions analysed below are idealized and do not take account of complex variations which occur in the ground"* and

*"Eventually it must be a matter for the engineer to judge what reliance to place on site investigation data and other engineering geology considerations, and thence to decide what factors of safety to employ".*



The authors note that the individual conclusions of James and Kirkpatrick (1980) and James (1981) assume:

(a) the relatively simple joint and "particulate" models show on Figure 3.31;
(b) the largest joint apertures for limestone, on Table 3.5, and
(c) the limiting seepage velocity of $3 \times 10^{-4}$ m/s for pure water through particulate or fragmented limestone, on Table 3.6.

With regard to (a) and (b) we note that at sites in faulted and weathered carbonate rocks the geological situation as exposed in detail at the excavated foundation surface and in tunnels, shafts or boreholes below it, can be very complex. In addition to features of the types on Figure 3.31, any or all of the following have been observed, often at great depths below ground surface:

- Open cavities or tunnels of all sizes, regular or irregular shapes, connected or disconnected;
- Cavities as above, partly or wholly filled with soils of variable erodibility;
- Beds or masses of variably weathered and erodible carbonate rock (see Section 3.7.1.2), and
- Beds or masses of weathered and erodible non-carbonate rock.

Achieving a satisfactory grout curtain can be extremely difficult or impossible in a foundation containing such features. During dam operation removal of soil or weathered rock by erosion could provide a flow path wide enough to cause ongoing solution of adjacent limestone. This type of process may have occurred at Hales Bar, Lar and Attaturk Dams (see Section 3.7.2.1 and Table 3.3).

With regard to (c), the maximum leakage rates shown on Table 3.3 suggest that rates much higher than $3 \times 10^{-4}$ m/s are/were being tolerated, at least locally, through the foundations of some of these dams. Such local flows may have high velocities and could be turbulent. It is noted also that Bozovic et al. (1981) and Riemer et al. (1997) foreshadow the possibility of future increases in leakages at Keban and Attaturk Dams.

However, as discussed previously in Section 3.7.2, many dams have been built in *Category O* limestone sites, with cement grouting for seepage control and have performed well for many years without excessive or increasing seepage. The authors accept that a good grouting program should be adequate to control seepage and prevent ongoing solution at many dams built on such carbonate rocks. However, it is recommended that those responsible for building any dam on these rocks should:

- Pay special attention to the design of monitoring systems (see Chapter 20) and
- Make adequate allowance (e.g. access and ability to lower storage levels) for additional grouting programs which might be required after first filling or later during the lives of the dams.

#### 3.7.6.2 *Category Y carbonate rocks*

The resistance of *Category Y* carbonate rocks to dissolution in fresh water has not been researched to the extent of the work done on *Category O* rocks by James and Kirkpatrick (1980) and James (1981). However, the following reasons suggest strongly that their potential for continuing dissolution, if present in a dam foundation, would be higher than that for *Category O* rocks:

- Their lower strengths and higher porosities;
- The field and laboratory evidence of their rapid solution and redeposition, presented in Section 3.7.7.2, and
- The experiences at Kopili, Montejagne, Perdikas and May dams (Table 3.4).

### 3.7.7 *Potential problems with filters' composed of carbonate rocks*

Most filters are well graded sands or gravels with few or no fines. They are compacted to 60% to 70% density ratio. Their moisture environments (in chimney zones, in particular) may vary widely. For example, in a filter, months or years of inundation and large or small flow rates may be followed by months or years of unsaturated, ranging to almost dry, conditions. Heavy rainfall at any time can cause water containing carbon dioxide to enter the upper part of a chimney zone. Carbon dioxide developed by the oxidation of methane from the groundwater or reservoir, organic acids derived from rotting vegetation or sulphuric acid derived from oxidation of sulphide minerals may also enter a filter zone. Possible effects of these events on a carbonate rock filter include:

(a) change of grading due to dissolution, or
(b) partial dissolution and recementation, or
(c) interlocking of grains due to pressure solution (see Figure 3.33).

Each of these effects and the likelihood of occurrence are discussed in Sections 3.7.7.1 and 3.7.7.2.

#### 3.7.7.1 *Category O carbonate rocks*

Dense, old carbonate rocks in the strong to very strong range have been crushed and processed to provide coarse and fine filter materials for many existing dams. The authors have found no report of a dam incident or failure due to malfunction of any of these filters.

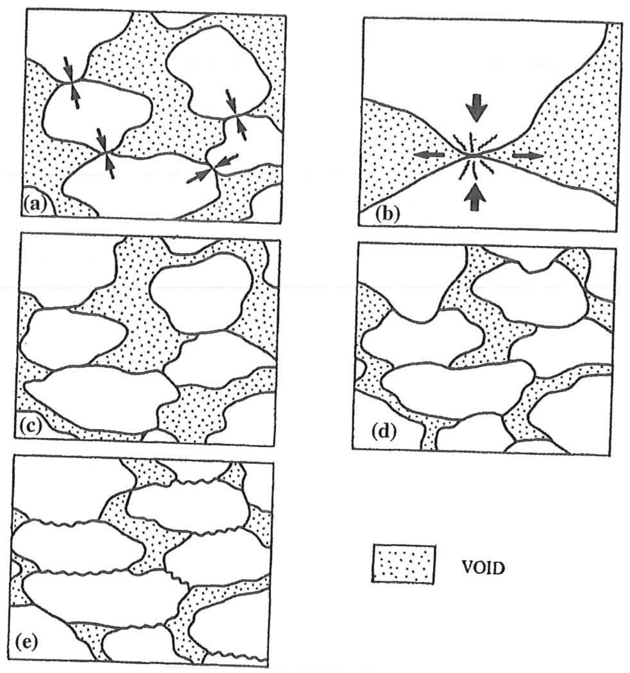



Figure 3.33. Progressive effects of pressure solution in granular materials (Harwood, 1988). (a) Point grain to grain contacts (arrowed), (b) Stressed grain to grain contact (large arrows) leading to formation of dislocations in crystal lattice and subsequent dissolution, with lateral fluid transport of solutes (small arrows), (c) Planar grain to grain contacts, (d) Interpenetrating grain to grain contacts, (e) Sutural grain to grain contacts.



However, not all dams have instruments sufficient to monitor the performance of all of their filters. Also the critical filters (Figure 9.2) may be seriously tested only in an emergency, e.g. when a crack develops in a dam core or the foundation beneath either of the dam shoulders is disrupted.

(a) *Change of grading due to dissolution.* The critical filter zones in most large dams are often less than 2.8m wide. According to Table 3.6, if pure water was to flow at a rate of 3 3 1024m/s through such a zone formed by 50 mm diameter calcite particles, it would lie entirely within the "solution zone". That is, the water would never become saturated with carbonate, and the particles would be progressively reduced in size, by dissolution. With much smaller particles, as in a fine filter zone, with acidic water and an aggressive reaction the size reduction might become significant during the life of a dam.
(b) *Partial dissolution and recementation.* There have been many accounts of recementation of crushed carbonate materials, due to acids formed by the oxidation of sulphide minerals (See Section 2.9.4). Several examples have been in South Australia where cemented zones have occurred in pavements, embankments and stockpiles formed by crushed rocks of Cambrian and Precambrian Ages. Tests have shown the cementing materials to be calcium sulphate (gypsum) in crushed marble and magnesium sulphate in crushed dolomites. Both the marble and dolomite have performed well as base courses in pavements.
    The minus 30 mm crushed marble is well graded and contains 5% to 10% fines (all calcite) with plasticity index of 2. Despite Los Angeles Abrasion Losses of 60% or more, its performance in pavements is remarkably good. Falling Weight Deflectometer tests on a sealed pavement built in 1998 showed its initial modulus to be 700 MPa. The same test after 4.6 years of service showed 1700 MPa. There was no rutting, permanent deflection or change in density (Andrews 2003). The authors consider that the formation and deposition of the gypsum cement, and possibly some associated interlocking of the marble particles due to pressure solution (Figure 3.33) have been the main causes of this increase in stiffness with time.
(c) *Interlocking of grains due to pressure solution.* Both gypsum and magnesium sulphate are highly soluble in water. This raises the following question. Assume that a chimney zone filter has become cemented by either of these during a period of very low and zero seepage. If now increased flow occurs through a crack in the core and cemented filter, how would this filter behave as its cement becomes redissolved? The results of testing by Hazell (2003) suggest that the filter might retain the crack, or "hang up". The test used was developed to assess the effects of chemical stabilising agents in base materials for unsealed roads. Samples are prepared by removal of the plus 2.36 mm fractions and compacting the remainder at OMC to 98% MMDD in 100 mm long by 100 mm diameter moulds. After removal from the mould samples are bench dried for 7 days. Each is then loaded by an annular weight of 1560 g, through which water drips on to it at a rate of about 400 ml/h. Hazell (2003) advises that during this test all other local non-cementing road base materials (most are crushed quartzites) collapsed within 24 hours, but the crushed marble remained intact, although completely saturated and soft to touch, for at least 460 hours. Assuming that by this stage the gypsum cement would have been completely dissolved, then the sample must have gained strength from some other process. The authors suspect that particle interlock may have developed due to pressure-solution at the grain contacts.

The process and effects of pressure solution in quartz sands in geological time frames are described by Dusseault and Morgenstern (1979), Palmer and Barton (1987) and Schmertmann (1991). Harwood (1983) discusses the role of this process in the reduction of intergranular porosity of sands. Pressure solution effects should proceed much more rapidly in carbonate sands than in quartz sands, because the solution rates of the carbonate minerals are much higher than that of quartz. Relatively inexpensive research (e.g. examination of thin sections cut through epoxy-impregnated old pavements) may show whether or not the effects are rapid enough to render filters composed of *Category O* carbonates ineffective.

#### 3.7.7.2 *Category Y carbonate materials*

(a) *Change of grading due to dissolution.* Sedimentary geologists consider that in fresh water high magnesian calcite is more soluble than calcite and aragonite (Prothero and Schwab 1996). Quantitative data on the solubility and solution rate of high magnesian calcite are not presently available. However, from the field and laboratory evidence discussed below, the authors believe that *Category Y* carbonates used as filter materials would suffer more rapid dissolution than equivalent *Category O* carbonates.

(b) *Partial dissolution and recementation.* Netterberg (1975) discussed very high CBRs yielded by pedogenic calcretes in Southern Africa and stated *"It is difficult to ascribe soaked CBRs much above those yielded by crusher-runs to anything other than cementation"*. He also included test results showing that the CBRs of some calcretes had more than doubled after a few cycles of wetting and drying. In Netterberg (1969) he described studies aimed at understanding how these calcretes were formed in soil profiles. He concluded that fine carbonate material in the soil is dissolved and redeposited as cement during changes in soil water suction and partial pressure of carbon dioxide. He also provided estimated rates of calcrete formation in nature, ranging from 0.02 mm/year to 1 mm/year (Netterberg 1969, 1978). In Netterberg (1971) he suggested that the apparent "self stabilisation" of calcretes used in pavements may be caused by similar (solution and redeposition) processes. The authors have seen many examples of recemented calcrete pavements in Australia.

Sterns (1944) and Tomlinson (1957) describe evidence of recementation of coral sands and gravels, where used as base materials in airfield pavements.

Coquina limestone of Tertiary age is used widely (when crushed) as base course for highways in Florida. This rock comprises corals, shell fragments and quartz grains. Graves et al. (1988) report that the strength increase which occurs with time in these pavements is due to dissolution and reprecipitation of fine carbonate particles. They support this claim using results of the LBR (limerock bearing ratio) test, a modified version of the CBR. Quartz sands were mixed in different proportions with carbonate sands containing less than 2% of fines. The samples were compacted into LBR moulds and tested after soaking continuously in water for periods of 2, 7, 14, 30 and 60 days. All samples with 40% or more of carbonate sand showed strength increases ranging from 23% to 65%, after the first 14 days of soaking. No further increase was recorded after this time. Graves et al. suggested that the very fine carbonate particles may have been "used up" by the end of 14 days and that the coarser grains were not providing cementing material as efficiently. Tests with crushed carbonate base course materials containing "slightly more carbonate fines" were tested similarly for 30 days and showed increases in strength up to that time.

McClellan et al. (2001) carried out further laboratory research into these effects but were unable to reproduce comparable strengthening of samples within curing times of up to 60 days.

(b) *Interlocking of grains due to pressure solution.* Shoucair (2003) has advised that the Florida Department of Transport has been adding coral sands to their road bases with significant strength gains. However he notes that the gains "practically disappear when the materials are soaked again" and considers that mechanisms described by Schmertmann (1991) are more likely causes of the initial strength gains than solution and recementation of the carbonate material.

One of the mechanisms described by Schmertmann was interlocking of grains due to pressure solution and the authors consider it possible that this may be the cause of the small remaining strength after resoaking.



Based on their experience with the gypsum-cemented marble (see Section 3.7.7.1) the authors suggested that the initial strength increase might have been caused by the presence of minute amounts of gypsum and that this could have been present in the coquina or in the waters used. McClellan (2003) has advised that the presence of gypsum is possible, even likely, during road construction in Florida. Traces of pyrites occur in some of the limestones and sea sprays (containing sulphates) occur widely. However he believes that gypsum is not a factor in their laboratory tests. SEM/EDX examination of the cemented areas of their specimens has never shown any significant sulphur.

Some very weak, porous calcarenites are sawn into blocks, which are then used for the walls of small buildings. Newly-sawn blocks are soft to touch and easily scratched, gouged or broken. After exposure to the weather (wetting and drying) for several years, the surfaces of the blocks become noticeably stronger and more durable. It appears that this "case-hardening" happens because carbonate derived from dissolution within the blocks becomes deposited near their surfaces. The Gambier Limestone (Tertiary Age) in South Australia is an example. Millard (1993) notes that "soft local limestone" on Malta, shows similar behaviour. Similar "case-hardening" effects can be seen in the faces of many cliffs formed by *Category Y* limestones, and carbonate-cemented sands.

The following conclusions can be drawn about *Category Y* carbonates:

1. When subjected to compaction, followed by soaking or wetting and partial drying, some of them have become strengthened by solution and recementation of carbonate grains in a few days to a few years. In most cases the above has involved some very fine carbonate material. In some cases most of the strength gains have been lost on re-soaking;
2. Exposed surfaces of some very weak, porous carbonate rocks become strengthened in a few years by solution and redeposition of carbonate;
3. In each of the above cases it is possible that the strengthening has been partly or wholly caused by gypsum.

The moisture environments in filter materials (in chimney zones, in particular) may at times be similar to those in which cementation was postulated, in 1 above. A carbonate filter zone would not normally contain significant amounts of fines. However the authors consider that the possibility exists that during the lifetime of a dam, solution and recementation and/or grain interlock from pressure solution, could render it cohesive and ineffective.

The authors consider that carbonate materials should not be adopted for filters until the possible long-term effects of dissolution have been assessed by geotechnical specialists.

### 3.7.8 *Suitability of carbonate rocks for embankment materials*

*Category O* carbonate rocks have been used widely and with apparent success as rockfill, random fill and riprap. However, there is some potential for deleterious effects, if sulphide minerals are present:

- in the carbonate rocks;
- in other materials in the embankment, or
- in the foundation or storage area.

Sulphuric acid formed by oxidation of the sulphides may attack the carbonate rock and form calcium or magnesium sulphates, which could become deposited in adjacent filter zones.

Carbonate rocks commonly occur in association with mudrocks containing sulphide minerals and carbonate rocks themselves often contain sulphide minerals. In Australia sulphide minerals can occur in all rock types and have been found at the sites of many dams. Quite small concentrations (much less than 1%) of these minerals can have deleterious effects.

The authors believe that it is best to avoid using carbonate rocks (of any age) as embankment materials, if sulphide minerals are present in any of the above ways, as the solution products could clog filters. In any rock or at any site, it can be difficult to determine the amount of sulphides present and their distribution.

### 3.7.9 *Suitability of carbonate rocks for concrete and pavement materials*

Dense carbonate rocks in the strong to extremely strong range have been used extensively for the production of aggregates for concrete, bituminous concrete and pavements. However, a few dolomitic rocks have been found to react with alkalis in Portland cement causing expansion and cracking effects like those from alkali-silica reaction. Experiences at sites where this has occurred are described by Luke (1963), Highway Research Board (1964) and Huganberg (1987). Guillott (1975, 1986) describes petrographic work on reactive carbonate rocks and concludes that only very fine grained dolomitic rocks containing some clay are likely to cause expansion.

Some carbonate rocks contain nodules or beds of chert (extremely fine grained or glassy silica). Alkali-silica reaction is likely to occur if these rocks are used as aggregate with high alkali cement.

The authors recommend that all carbonate rocks intended for use in concrete be assessed for reactivity.

### 3.7.10 *Stability of slopes underlain by carbonate rocks*

Natural landsliding is not common in areas underlain by pure carbonate rocks. In weathered, solution affected carbonate rocks it is common to find that joints, faults and bedding partings have been partly or wholly "healed" by redeposited calcite. This along with their inherently high frictional strength of joints seems likely to be the reason for the low frequency of landslides.

In the experience of the authors most slides in carbonate rocks have occurred along interbeds of mudstone or shale. Figures 3.34 to 3.36 show a landslide of approximately 2500 million $m^3$ in folded limestone of Tertiary Age in Papua New Guinea. The slide is believed to have occurred into an abandoned valley of the Mubi River, due to daylighting of a thin mudstone bed within the limestone (Figure 3.36). The slide was probably triggered by earthquakes associated with fault movements, which displaced the Mubi River about 400 m laterally, and by continued uplift and tilting of the limestone to the south of the fault.

The Bairaman landslide, also in Papua New Guinea, occurred in a 200 m thick horizontal bed of limestone (King et al., 1987) and its horizontal basal failure surface was semicircular in plan, covering about 1 $km^2$. Its basal surface must have been a mudstone bed.

In detailed studies of carbonate rocks in the Vaiont landslide Hendron and Patton (1985) have shown that clay-rich units and clay are present along and near most of the failure surface.

James (1983) draws attention to situations in Sri Lanka where solution of near-horizontal limestone beds near valley floors has caused collapse, undercutting and landsliding in



Figure 3.34. Air photo showing the scar of an old landslide in folded limestone, near the Mubi River, Papua New Guinea.

overlying beds. Retreat of these beds has produced broad valleys bounded by steep escarpments, with hanging tributary valleys. James considers these almost "glacial-like" valleys to be useful indicators of the presence of limestone in areas devoid of outcrop.

### 3.7.11 *Dewatering of excavations in carbonate rocks*

Excavations below the water table in carbonate rocks containing cavities are likely to require more continuous pumping at higher rates than equivalent excavations in non-soluble rocks. Also because the sizes and distribution of cavities are usually so variable and irregular, a number of borehole pumping tests at carefully selected locations may be needed to get a reliable estimate of the inflow rates.

### 3.7.12 *Carbonate rocks – check list of questions*

- Category *O or Y*?
- Cavities, air-filled or water-filled?
- Cavities, soil-filled?

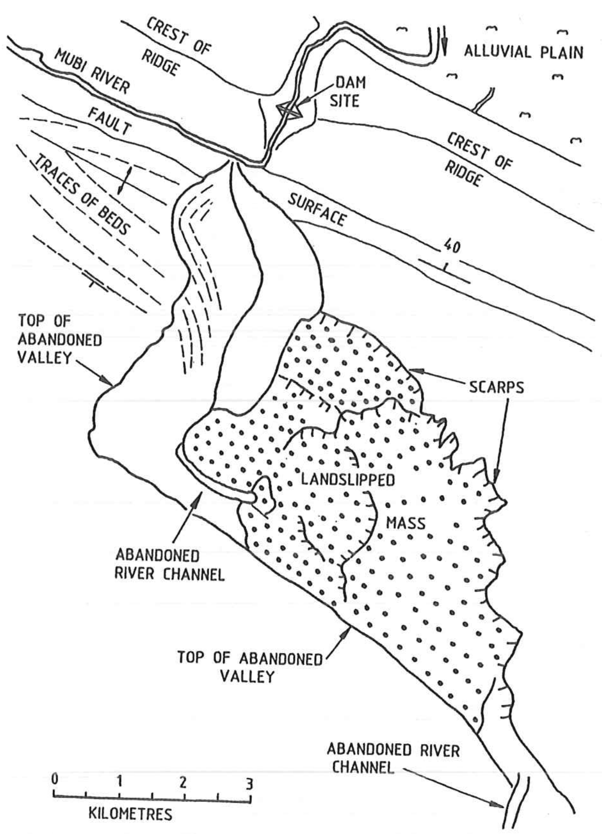



Figure 3.35. Geological plan of the area shown on Figure 3.34.

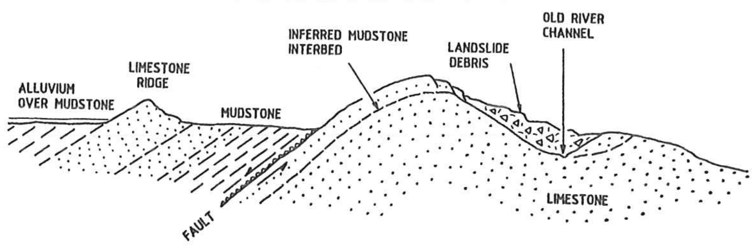



Figure 3.36. Sketch section through the area shown on Figure 3.35.

- Collapse of cavities?
- Extremely irregular, often pinnacled surface of fresh rock?
- Sharp boundary between residual soils and fresh rock?
- Strong rock around solution tubes and cavities in weak, porous rocks?
- Solution cavities in altered carbonate rocks or metamorphosed impure carbonate rocks?
- Very weak, low density, erodible weathered materials?
- Extremely high permeabilities?
- Extreme variations in permeability?
- Possible deep, major leakage paths out of reservoir?



- Presence of sinkholes, exposed or concealed?
- Composition and pH of the groundwater and reservoir water?
- Presence, amounts and distribution of any sulphide minerals?
- Potential for dangerous ongoing solution in the dam foundation?
- Suitability for use for embankment materials?
- Suitability for use in concrete and pavements?
- Alkali-carbonate reaction?
- Chert present: Alkali-silica reaction?
- Shaley (argillaceous) rocks: Durability?
- Unstable slopes, where interbeds of mudrocks are present?

## 3.8 EVAPORITES

The common evaporites, gypsum ($CaSO_4.2H_2O$) and halite (NaCl) are formed in arid areas by evaporation from inland seas, from inland and coastal salt lakes and tidal flats (Stewart, 1963; Murray, 1964). They can occur as individual beds in sedimentary rock sequences, often in association with or interbedded with carbonates. They occur also as matrix, cement, nodules, veins or joint fillings, in mudrocks or sandstones. Anhydrite ($CaSO_4$) is a less common evaporite. Most anhydrite is believed to be formed from gypsum, when it is buried at depths greater than 150 m and its chemically attached water is removed due to overburden pressure and heat. Anhydrite also occurs as nodules or infilling cavities in limestone and dolomite (Murray 1964; Brune, 1965).

The evaporites are much more soluble than the carbonates and only outcrop in arid regions. Sequences containing evaporites show a wide range of solution effects related to present and/or past groundwater levels. Brune (1965), Bell (1983b, 1993), Cooper (1988), Thompson et al. (1998), Hawkins and Pinches (1987), James (1992, 1997), Hawkins (1998) and Yilmaz (2001) describe such effects and related ground engineering problems. These include:

- karst topography with extensive systems of caves, tunnels, depressions, chimneys and sinkholes;
- ongoing subsidence, on small and large scales; very slow or sudden collapse;
- leakage through or into solution cavities;
- ground weakening and/or increasing permeability due to ongoing solution;
- heaving ground, due to growth of gypsum crystals;
- explosive heaving/uplifting of ground due to the hydration of anhydrite to become gypsum. Brune (1965) describes how creek channels in Texas were cracked open and uplifted several metres along distances of up to 300 m during such explosions.

### 3.8.1 *Performance of dams built on rocks containing evaporites*

The authors know of no dam built at a site containing thick beds of halite. Many dams and reservoirs have been built successfully on sites containing gypsum. A few have later suffered some degree of distress, caused by ongoing dissolution of this material from their foundations. James and Kirkpatrick (1980) refer to Poechos Dam in Peru, founded on clay shales with gypsum, noting that seepage from its right "dyke" was saturated in calcium sulphate and also contained 3.5% of sodium chloride. Notable embankment dam examples in the USA include McMillan and Cavalry Creek Dams (Brune, 1965), Tiber Dam (Jabara and Wagner, 1969). San Fernando Dam (concrete gravity) was affected by rapid solution of gypsum cement from very weak conglomerate (Ransome, 1928; Jansen, 1980).

A more recent example with a gypsum-related issue is Caspe Dam in Spain. Cordova and Franco (1997) cite dissolution of gypsum in the foundation as the cause of a large leakage and damage to this 55 m high embankment dam in 1989 during its second filling. Another is Tarbela Dam in Pakistan, where the right abutment rocks are intensely fractured and include gypsum and "sugary" limestone which is friable and erodible. Grouting has been only partially effective in controlling flows which have high contents of dissolved solids. Additional drainage adits have been required (Amjad Agha, 1980).

Reports on the failure of Quail Creek Dike (James et al., 1989; O'Neill and Gourley, 1991) provide useful details on the weathered profile developed on the interbedded dolomite, siltstone and "silty gypsum" which formed the foundation of that 24 m high embankment dam. Its failure was not related to any post-construction solution effects, but was due largely to the failure during design to recognize and take into account the gaping joints in the near-surface rock produced by the past solution of gypsum.

### 3.8.2 *Guidelines for dam construction at sites which contain evaporites*

Predictive models derived by James and Lupton (1978) for the rates of dissolution of gypsum and anhydrite have been discussed briefly above in Section 3.7.4. James and Lupton also provide guidelines for the investigation of dam or reservoir sites where these minerals are present and indicate the kinds and levels of risks associated with construction and operation at such sites. They point out that it is important to know the chemical compositions of the waters involved, because the solution rates of both gypsum and anhydrite are increased by the presence of sodium chloride, carbonate and carbon dioxide. They note that conglomerates which are cemented by gypsum or anhydrite "can produce a material which is potentially very dangerous" if small proportions of the cement are removed by solution. They conclude that the risks of accelerating solution effects are higher with anhydrite than with gypsum and that an "efficient cutoff" is the only practical method to reduce seepage velocities to values low enough to provide "complete safety" against solution effects in "massive anhydrite". They also warn about the possibility of anhydrite converting to gypsum and expanding, with the potential for heave.

James and Kirkpatrick (1980) confirm the conclusions of James and Lupton and provide further predictive data for gypsum and anhydrite and comparable data for halite (see Tables 3.5 and 3.6). They stress the need for chemical tests on drilling water and cores, when halite is suspected, and special sampling procedures if halite is found. They conclude "It would be most unwise to build a dam on massive halite. Unconsolidated strata containing sodium chloride may be unavoidable and control measures are feasible, if costly".

The authors acknowledge the valuable work carried out by James and Lupton (1978), James and Kirkpatrick (1980) and James (1992) and endorse their views, subject to the following:

- We would recommend extreme caution to those considering dam construction at sites where thick beds of anhydrite occur at depths shallower than 150 m;
- As for sites on carbonate rocks (see Section 3.7.6) we would have some reservations about the effectiveness of cement grouting at sites containing thick beds of evaporites severely affected by past weathering and solution. Special attention would be needed to seepage monitoring and facilities for re-grouting during operation of the dam;
- The possibility that filter zones could become cemented with gypsum needs to be considered;
- It is our view that engineers intending to build at sites containing evaporites would be wise to get advice from persons with special knowledge of the evaporite minerals, and from others with past experience of construction at such sites.



### 3.8.3 *Evaporites – checklist of questions*

- Cavities, air-filled or water-filled?
- Cavities, soil-filled?
- Collapse of cavities – subsidence?
- Ground weakening due to ongoing solution?
- Increasing permeability due to ongoing solution?
- Heave due to growth of gypsum crystals?
- Large scale heave due to hydration of anhydrite?
- Chemical composition of groundwaters/reservoir waters?
- Presence of halite – chemical tests?
- Possibility of cementation of filter materials, by gypsum?

## 3.9 ALLUVIAL SOILS

For the purposes of this discussion "alluvial soils" includes soils which have been deposited in the channels and flood-plains of rivers and in lakes, estuaries and deltas. These soils are characterised by great variability, both vertically and laterally and can range from clays of high plasticity through to coarse sands, gravels and boulders.

Detailed sedimentological studies in recent years have provided one or more "facies models" for the sediments (soils) deposited in each of the above environments. A facies model includes block diagrams which summarise the compositions and configuration of the bodies of soil deposited in a particular environment, together with an indication of the processes by which they were formed. Figure 3.37 is a block diagram showing key features of the "meandering river" model, which will be the only one discussed in detail here. Further information about meandering rivers and details of the deposits formed in other environments can be found in Leopold et al. (1964), Leeder (1982), Selley (1982), Lewis (1984) and Walker (1984).

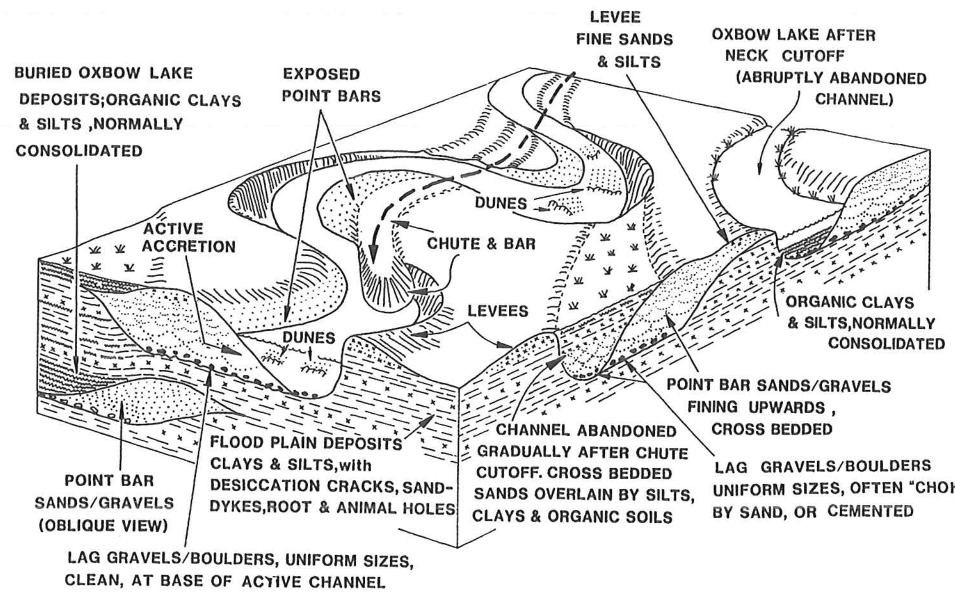



Figure 3.37. Schematic view of soils deposited by a meandering river (based partly on Figure 1 of Walker, 1984).

The main types of deposits shown on Figure 3.37 are as follows:

- Lag deposits, usually gravel or boulders, occur along the base of the river channel and are moved only during peak flood times. They are usually uniform sized and in the active river channel may have very high voids ratio and permeability. Where preserved at the bases of abandoned or buried channels their voids may be "choked" by sand or by fines.
- Point bar deposits, usually sands and gravels, are deposited on the insides of bends in the stream. During normal flows these materials occur above the lag deposits along the whole of the channel with their upper surfaces in the form of migrating dunes. The cross-bedding seen in the bar deposits results from preservation of some of these dunes. The point bar deposits are usually coarser at depth, becoming finer towards the top.
- Levees of fine sands and silts are formed along the top of the river banks where these coarser materials are deposited more quickly than the fine silts and clays when floodwaters overtop the banks.
- Flood-plain deposits are usually fine silts and clays, deposited in thin horizontal layers during floods. In situations where the soils dry out between floods, desiccation cracks are formed and these may be preserved as sand infilled joints if wind-blown or water-borne sand is deposited in them. Small tubular holes left by burrowing animals or decomposing vegetation are also common in floodplain deposits. These may be preserved open or else infilled by sand or fines.
- Oxbow lake deposits occur in lakes formed during floods when parts of the meandering channel are cut off from the stream. The nature of the deposits depends on whether the channel is abandoned slowly (chute cutoff) or abruptly (neck cutoff) as shown. In each case the fine-grained soils are usually near normally consolidated and are at least partly organic due to the presence of rotted vegetation.

Not shown on Figure 3.37 are bar deposits which can be seen at low flows along straight parts of meandering streams and also in the channels of "braided" streams. Such bars may migrate quite rapidly with aerial photographs taken 10 years apart showing significant changes in the stream bed geometry.

Cary (1950) reports that gravel bars in several fast-flowing rivers in U.S.A. contain elongated lenticular deposits of essentially uniform-sized "open-work" gravel. He reports many open-work gravel lenses in glaciofluvial deposits in the north-western U.S.A., and comments on the large voids and extremely high permeabilities of these materials. The authors have seen open-work gravels and cobbles in which the large voids have become "choked" with sand, which clearly must have migrated into the voids after the original deposition of the gravels.

Cary considers that open work gravels are probably formed in fast-flowing rivers at the downstream ends of rapidly aggrading bars. He suggests that in these situations eddies sometimes occur, which remove finer gravels and sand, leaving only the coarse materials which form the open-work deposits.

In arid or semi-arid climates where stream flows are intermittent, it is common to find cemented layers in the lag and lowest bar deposits of meandering steams and of other e.g. braided streams. This cementation occurs as the waters dry up and the most common cements include gypsum, calcite and limonite. Cementation is relatively common also in the floodplain deposits.

It is not uncommon to find timber, in some cases the remains of large trees, buried in channel or floodplain deposits. The timber is often well preserved, particularly where groundwaters are highly saline. In some situations the timber is partly or wholly rotten and may have left gaping voids in the alluvial deposit.

It is difficult to generalise about the properties of alluvial soils, because of the extremely wide range of soil types. The following are some observations which may be taken as a general guide.



### 3.9.1 *River channel deposits*

The sands, gravels, cobbles and boulders are often highly permeable, particularly in the horizontal direction. Layers, often thin, of finer or coarser materials cause marked differences between vertical and horizontal permeability. This is shown diagrammatically in Figures 3.37 and 10.2. Clean gravels can be interlayered with sands or sandy gravels, giving overall horizontal permeabilities 10 times to 1000 times the overall vertical permeability. The relative density of such deposits is variable, but the upper few metres which are most affected by scour and redeposition during flooding, are likely to be loose to medium dense and, hence, will be relatively compressible and have effective friction angles in the range of 28° to 35°. Deeper deposits are more likely to be dense, less compressible and have a high effective friction angle.

### 3.9.2 *Open-work gravels*

At the 143 m high, 2740 m long Tarbela Dam in Pakistan extensive deposits of open-work gravels occur in the 190 m deep alluvium which forms the foundation for the embankment. The alluvium comprises sands, open-work gravels and boulders and boulder gravels in which the voids are sand-filled (i.e. extremely gap-graded materials). The design allowed for underseepage to be controlled by an impervious blanket which extended 1500 m upstream from the impervious core. The blanket ranged in thickness from 13 m near the upstream toe of the embankment to 1.5 m at its upstream extremity. For several years after first filling (1974) many "sink-holes" or graben-like craters and depressions appeared in the blanket, apparently due to local zones of cavitation within the underlying alluvium. Some of the sinkholes were repaired in the dry and others by dumping new blanket material over them through water using bottom-dump barges. The local collapse zones which caused the sinkholes are believed to have formed when excessively high flow rates through open-work gravels caused adjacent sandy layers to migrate into their large voids.

### 3.9.3 *Oxbow lake deposits*

Where clays, silts and organic soils deposited in oxbow lakes have not dried out they are near normally consolidated and may be highly compressible. McAlexander and Engemoen (1985) describe the occurrences of extensive oxbow lake deposits up to 5 m thick in the foundation of the 29 m high Calamus Dam in Nebraska, USA. These deposits comprised fibrous peat, organic silty sands and clays and were highly variable in thickness and lateral extent. Testing showed that the peat was highly compressible. Because of concern about differential settlements and cracking in the embankment, the organic materials were removed from beneath the impervious core and from beneath extensive parts of the shoulders.

### 3.9.4 *Flood plain, lacustrine and estuarine deposits*

The clays and silts in these deposits are likely to show pronounced horizontal stratification, with each flood or period of deposition resulting in an initially relatively coarse layer fining upwards as the flood recedes. This may result in marked anisotropy in permeability, with the horizontal permeability being 10 times or even 100 or 1000 times the vertical permeability.

The permeability of these deposits is often increased by the desiccation cracks, sand filled cracks, fissures and holes left by burrowing animals and rotted vegetation. Where such defects have been backfilled by clay soils, the permeability of the mass can be decreased.

Where desiccated the clay soils are overconsolidated and their shear strengths are affected by the presence of fissures which are often slickensided.

In environments where the water table has remained near the surface, e.g. coastal estuaries, the flood-plain soils may be very soft clays, which are near normally consolidated except for an overconsolidated upper 0.5–2 m. In these cases the soils at depth may have a very low undrained shear strength and very high compressibility.

### 3.9.5 *Use of alluvial soils for construction*

The filters and concrete aggregate for many dams are obtained from alluvial sand and gravel deposits. In many cases the strict grading requirements and need for low silt and clay content (usually less than 5% or 2% passing 0.075 mm) necessitate washing, screening and regrading. The source of the sediments will have a marked effect on their durability. For example, sand and gravel in a stream fed from areas partly underlain by siltstone is likely to have gravel size particles which will break up readily, rendering the gravel unsuitable for filters or concrete aggregates, whereas those originating from areas underlain by granite, quartzite or other durable rocks are more likely to be suitable.

Alluvial clays, sandy clays and clayey sands (including a proportion of gravel in some cases) can be suitable for earthfill zones in dam construction. Because of their likely variability, the deposits need careful investigation to delineate suitable areas. Borrowing with a shovel and truck operation is sometimes necessary to ensure adequate mixing. Some Australian dams in which alluvial materials have been used for earthfill core zones include Blue Rock, Cairn Curran, Buffalo, Eildon (Victoria), Blowering (N.S.W.), Bjelke-Peterson and Proserpine (Queensland), Hume (NSW – Victoria).

### 3.9.6 *Alluvial soils, list of questions*

- Vertical and lateral variability related to deposition conditions?
- Lenticular deposits of open-work gravels with extremely high permeability?
- Anisotropy due to layering?
- High kH : kV ratio?
- Oxbow lake deposits, compressible organic soils?
- Cracks, fissures, holes after rotting vegetation or burrowing animals, all either open or backfilled?
- Cemented layers?
- Buried timber, rotten or preserved, large voids?

## 3.10 COLLUVIAL SOILS

### 3.10.1 *Occurrence and description*

Included under this heading are all soils which have been eroded and deposited under gravity forces, often with the aid of water flow. They include slopewash, scree (talus), and landslide debris. The soils range from high plasticity clays through to boulder talus deposits, but are characterised by being mixtures of particles of contrasting sizes e.g. clays with embedded gravel and boulders in landslide colluvium and clayey gravelly sand slopewash deposits. They are also commonly variable within each deposit. Figures 3.38 and 3.39 show typical environments in which scree and slopewash deposits are formed.

#### 3.10.1.1 *Scree and talus*

These are deposits of rock fragments which detach from cliffs or areas of steep outcrops and fall by gravity and roll/slide downslope. The upper scree slopes are composed of smaller rock fragments and usually are at slopes of 35° to 38°; the toe of the slope usually



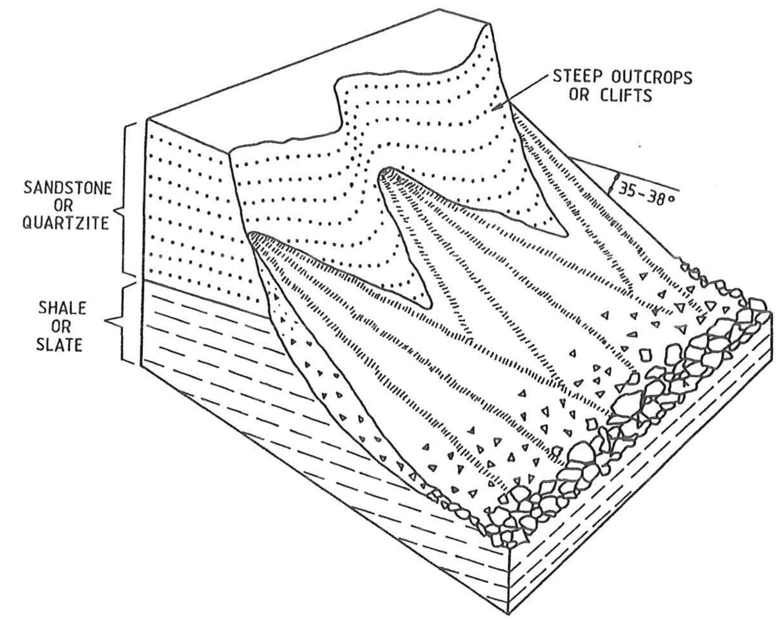



Figure 3.38. Schematic view of scree deposits.

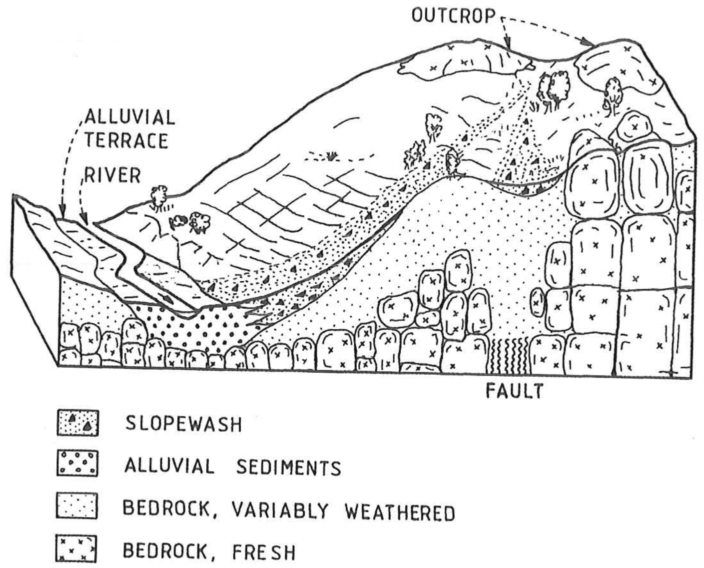



Figure 3.39. Schematic view of residual, slopewash and alluvial deposits.

comprises large blocks at flatter angles. The deposits are not water-sorted. They are usually very loose (low bulk density) and just stable at the natural angles. When the deposits contain 30% or more of fine-grained soil, they are called talus.

Selby (1982) provides a more detailed discussion on the variety of processes of formation of scree and talus and on the range of fabrics resulting from this. Some talus deposits show poorly developed soil profiles near the surface or at intervals at depth. These indicate periods during which weathering of the deposit has been proceeding at a faster rate than accumulation of new rock fragments.

It is not uncommon to find timber embedded in scree, either rotted or in a preserved condition.

#### 3.10.1.2 *Slopewash soils*

Slopewash soils are admixtures of clay, sand and gravel which have been moved downslope by the combined actions of soil creep (due to gravity forces) and erosion by water. The thickest deposits are developed in depressions or gullies as shown on Figure 3.39. Near the base of steep slopes slopewash soils often overlie or are intertongued with alluvial deposits (also shown on Figure 3.39).

In cold climates (see Section 3.12) freezing and thawing of the ground can be a major contributing factor in soil creep and deep slopewash deposits are common.

Slopewash soils sometimes show indistinct bedding parallel or non parallel to the ground surface. Slopewash usually has low density and often exhibits tubular voids left by rotted vegetation or roots or burrowing animals or caused by erosion of fines from within the deposit.

#### 3.10.1.3 *Landslide debris*

Landslide debris can range from high plasticity clay through to silty sand from ash flows or sand/gravel/boulder soils resulting from avalanches. In most cases the soils are very variable, vertically and laterally, and it is not uncommon to find large boulders embedded in a clay matrix.

Timber (the remains of trees) is often present in modern landslide debris. It may be well preserved or rotting. Voids left by rotted timber are sometimes found.

Open cracks and irregular voids are often found in landslide debris, particularly where the debris has resulted from or has been affected by modern slope movements.

At sites where landslides have dammed and diverted pre-existing rivers it is common to find landslide debris overlying river alluvium.

Deposits of landslide debris are often underlain by a sheared or slickensided zone (the slide surface) and there may be several sliding and shear surfaces at other levels within the debris. In many cases the main slide surface may be in a zone of material which appears to be residual soil or extremely weathered rock and is characterised by a higher clay content than that of most of the debris.

High groundwater tables are common in landslide debris, but this is not always the case.

### 3.10.2 *Properties of colluvial soils*

As for alluvial soils, it is difficult to generalise because of the extremely wide range of soil types. Some general characteristics which may be present are:

#### 3.10.2.1 *Scree and talus*

These materials are likely to be highly permeable, and compressible. As they are sorted they are likely to be poorly graded.

As these materials occur close to their natural angle of repose, excavation into scree or talus slopes usually causes ravelling failures extending upslope. Entry of excessive water (e.g. by discharge from roads) into talus materials can cause them to develop into debris-flows.

#### 3.10.2.2 *Slopewash*

These soils may be more permeable than expected from their soil classification, reflecting the presence of voids and loose structure. They are also likely to be relatively compressible. Many slopewash soils are highly erodible.



Where they occur on steep slopes (e.g. as in Figure 3.39) slopewash deposits are often only just stable. Construction activities which cause such deposits to be over-steepened or to take up excessive amounts of water can result in landsliding.

#### 3.10.2.3 *Landslide debris*

Many landslide debris soils have relatively low permeabilities, but their mass permeabilities may be high, due to the presence of cracks resulting from sliding movements. The shear strength of the colluvium is often reasonably high, but slide surfaces at the base and within the colluvium will be at or near residual. Almost invariably, the soil at the base of the slide is not the same as the slide debris, so shear strength tests on the slide debris can be misleading and usually overestimate the strength. Where the colluvium is derived from fine grained rocks such as shale, siltstone or claystone, the weathered rock underlying the colluvium often has higher permeability than the colluvium, an important point when considering drainage to reduce pore pressures and improve stability.

Landslide debris deposits are often only marginally stable and slope instability may be initiated by minor changes to the surface topography or to groundwater conditions (see Section 2.10.1).

### 3.10.3 *Use as construction materials*

Landslide debris and slopewash were used for the impervious core of the 161 m high Talbingo Dam, see Hunter (1982) and Section 2.10.3. The landslide debris, composed mainly of extremely weathered andesite, was used in most of the core. The slopewash, higher plasticity material derived from the landslip deposit, was used as core-abutment contact material.

Slopewash derived from extremely weathered granitic rocks was used for core-abutment contact material in the following dams, for which the parent extremely weathered granite formed the remainder of the core.

- Eucumbene (N.S.W., Australia);
- Dartmouth (Victoria, Australia);
- Thomson (Victoria, Australia);
- Trengganu (Malaysia).

Slopewash derived from extremely weathered rhyodacite was used for the core at Tuggeranong Dam (A.C.T., Australia).

When considering the possible use of landslide debris as earthfill, the critical issues are the potential variability of the soil and the possible need to remove large boulders and cobbles. Also the possibility of renewed slope movements must be considered, where the deposits occur on sloping ground. High groundwater levels and resulting wet conditions may create further difficulties.

### 3.10.4 *Colluvial soil – list of questions*

(a) Scree and talus:
   - High permeability and compressibility?
   - Timber debris, rotted or preserved?
   - Potential for instability or debris-flow?

(b) Slopewash:
   - Tubular voids causing high mass permeability?
   - Compressible?
   - Erodible?

- Potential for slope instability?

(c) Landslide debris:
- Variability in composition and properties – laterally and vertically?
- Boulders?
- Large voids?
- Gaping or infilled cracks?
- High compressibility?
- Timber, rotting or preserved?
- High permeability?
- High water tables – wet conditions?
- Old slide surfaces of low strength at the base, or at other levels in the deposit?
- Potential for renewed sliding movements?

## 3.11 LATERITES AND LATERITIC WEATHERING PROFILES

Townsend et al. (1982) provide the following definition of laterite:

*"Laterite refers to varied reddish highly weathered soils that have concentrated oxides of iron and aluminium and may contain quartz and kaolinite. Laterite may have hardened either partially or extensively into pisolitic, gravel like, or rock-like masses; it may have cemented other materials into rock-like aggregates or it may be relatively soft but with the property of self-hardening after exposure".*

### 3.11.1 *Composition, thicknesses and origin of lateritic weathering profiles*

The following features of lateritic weathering profiles, some of which are shown also on Figure 3.40, are based mainly on Selby (1982).

| Thickness (m) | Description |
|---|---|
| 0 to 2 | Soil zone, often sandy and sometimes containing nodules or concretions; this may be eroded away. |
| 2 to 10 | Ferricrete or alcrete crust of reddish or brown hardened or slightly hardened material, with vermiform (or vermicular) structures (i.e. having tube-like cavities 20–30 mm in diameter) which may be filled with kaolin; less cemented horizons may be pisolithic (i.e. formed by pea-sized grains of red brown oxides). |
| 1 to 10 | Mottled zone; white clayey "kaolinitic" material with patches of yellowish iron and aluminium sesquioxides. |
| Up to 60 but often <25 | Pallid zone; bleached kaolinitic material; the distinction between the mottled and pallid zone is not always apparent and they can be reversed; silicified zone which may be hardened (i.e. silcrete on Figure 3.40). |
| 1 to 60 | Weathered rock showing original rock structures. |

The "crust" materials are termed ferricrete when they consist mainly of iron oxides. The crust can be either gravel or rock ranging from very weak to very strong, often requiring blasting for its excavation. The very weak rock materials often become stronger when exposed to the weather.

Laterites are believed to have been formed under tropical or sub-tropical climatic conditions and most laterites of Tertiary Age and younger occur in tropical or sub-tropical areas on both sides of the equator. Lateritic profiles are found also within rocks of Mesozoic and Palaeozoic Ages, mostly at higher latitudes, where their presence indicates ancient tropical conditions which can be explained by continental drift (Bardossy and



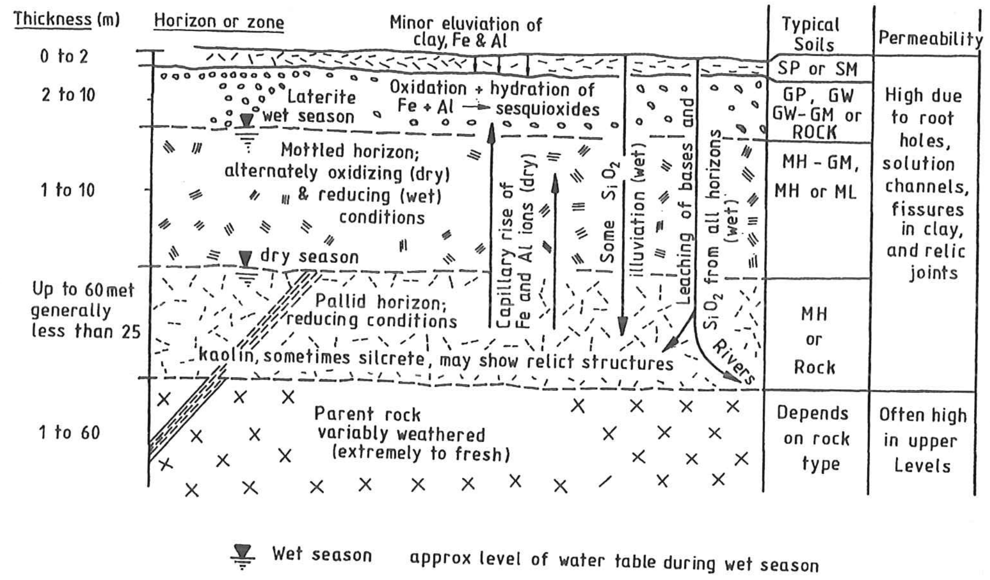



Figure 3.40. Diagram showing the lateritic weathering profile of Selby (1982) and some of the processes involved in its development (by permission of Oxford University Press).

Aleva, 1990). Laterites have been developed on the full range of common igneous, metamorphic and sedimentary rocks (Gidigasu, 1976).

There are several theories about the formation of laterites (Bardossy and Aleva, 1990; McFarlane, 1976) but most include the influence of a fluctuating water table to allow solution and transfer of soluble silica, iron and aluminium ions, resulting in iron and aluminium oxides accumulating in the upper part of the profile (Figure 3.40).

Lateritic profiles may be much shallower than described above e.g. in the Ranger Mine area in northern Australia and in many other exposures in Australia and south-east Asia they are less than 5 m thick. Some of these shallow laterite profiles are of detrital origin i.e. they comprise ferricrete and/or alcrete gravels which have clearly been eroded from an earlier laterite weathering profile and redeposited. These "reworked laterites" show varying degrees of re-cementation and may or may not be underlain by mottled and pallid zones.

Figure 10.3 shows some common features of lateritic profiles in valley situations in northern and western Australia.

### 3.11.2 *Properties of lateritic soils*

The most abundant soils in the mottled zone are usually clays, sandy clays or gravelly sandy clays, which behave as soils of medium to high plasticity, but which usually plot below the "A" line in the Casagrande classification chart. Strictly speaking they are therefore classified as silts according to the Unified Soil Classification. This behaviour is a result of the presence of allophane, kaolin, gibbsite, bauxite and often halloysite.

Other clays have been found in some laterites, for example Gordon (1984) records montmorillonite and illite in profiles developed over dolerite bedrock at Worsley, Western Australia. However this would not be common. Gordon and Smith (1984a,b) describe the results of field and laboratory tests on these and other laterite soils in that area.

As indicated on Figure 3.40 *in situ* laterite profiles are often highly permeable. Many of the structural features which cause the high permeability are near-vertical. In the upper zones vertical tubes, called "channels" or "drains" may have been formed by the decomposition of roots. These may be open or infilled with sand. The mass permeability in these zones may be $10^{-2}$ m/sec to $10^{-4}$ m/sec. These near-vertical features are not located readily by conventional drilling and water pressure testing and so in these cases the permeability is best determined by vertical infiltration tests involving relatively large test areas.

### 3.11.3 *Use of lateritic soils for construction*

Lateritic soils usually make excellent earthfill construction material. The most notable case of the use of lateritic soil was in Sasumua Dam, where Terzaghi (1958) showed that despite its apparently peculiar classification properties (i.e. plotting below the "A" line), the lateritic soil was an excellent dam building material. When used as fill, laterite soils are characterised by high effective friction angle and medium to low density and permeability. In most cases they are readily compacted despite often having high and poorly defined water content. For example at Sirinumu Dam, lateritic clays were readily compacted at water contents between 40% and 50%. However, some particularly silty laterites with high halloysite contents can be difficult to compact.

The ferricrete gravels and weak rocks in the near-surface crust zone are used in lateritic areas throughout the world as base or sub-base material in pavements for roads and airstrips. Strongly cemented rock from this zone has also been used successfully as rockfill and rip-rap.

In the laterite and mottled zone soils are usually non dispersive and in the authors' experience, very resistant to erosion *in situ* and recompacted.

### 3.11.4 *Karstic features developed in laterite terrain*

Sinkholes similar to those seen in karstic limestone are known to occur also in some laterite profiles developed on non-carbonate rocks.

Twidale (1987) describes sinkholes up to 50 m diameter and 15 m deep on the Western Stuart Plateau in Northern Territory, Australia. The plateau is underlain by a sequence of siltstones and quartzites, ranging from 40 m to 230 m in thickness, which is underlain by limestone. The laterite profile is developed at the top of the non-carbonate sequence and may be 20–30 m deep. Twidale (1987) suggests that where the sequence is thin some of the sinkholes may have formed by the lateritic materials collapsing directly into voids in the underlying limestone. However he believes that they most probably developed as follows:

- Voids were formed within the lateritic profile by the removal of silica and silicate minerals by solution, these being carried down joints, deeper into the sequence, and
- The ferricrete cap collapsed into the voids.

Twidale believes that the subsurface voids developed in late Tertiary time and that collapse of the ferricrete cap into the voids has continued intermittently since that time. He provides details of the youngest sinkholes, formed in 1982, and describes how many older ones are much modified by sidewall collapse and infilling with slopewash and alluvial soils.

Twidale (1987) refers also to karstic features in laterite developed on peridotite (ultrabasic rock) in New Caledonia. The karstic features in that area occur in broad plateaus surrounded by steep ridges which are remnants of relatively fresh peridotite. They include:

- internal drainage into sinkholes;
- recently active sinkholes up to 15 m wide, and
- swamps, believed to represent collapsed areas now filled with sediment and water.



Many of the sinkholes occur in lines above steeply dipping defect zones in the peridotite. The collapses appear to have occurred progressively into voids formed where silica and silicates have been removed by both dissolution and erosion and carried away in near-horizontal and near-vertical flow paths (widened defects) within the defect zones.

The authors suggest that, if such features are evident, investigations should be carried out to establish the mechanism and the potential effect on the dam or its storage.

### 3.11.5 *Recognition and interpretation of silcrete layer*

As shown in Figure 3.40, the silcrete formed by deposition of silica in the pallid horizon can contain "relic" structural features, e.g. bedding or foliation which were present in the original rock before it was weathered. It is important that this is understood, to avoid errors in interpretation of drill cores and exposures in excavations. For example, a typical silcrete layer is very strong rock, similar to quartzite and is near-horizontal. It is usually underlain by weathered rock with soil or very weak rock properties. If the silcrete was formed within weathered material which was previously vertically bedded rock, it will appear to show vertical bedding. It might then be misinterpreted as part of a vertical layer of quartzite bedrock, continuing to depth.

This type of situation was found during the raising of Hinze rockfill dam in Queensland (see Fell et al., 1992, Page 122). Steeply dipping beds of open-jointed and cavernous "quartzite" were found to grade downwards into dolomite and limestone. It is believed that silica released during lateritic weathering of adjacent metamorphic rocks had progressively replaced the uppermost 40 m or so of the dolomite and limestone, to form a cavernous silcrete horizon.

### 3.11.6 *Lateritic soils and profiles – list of questions*

- Variable laterally and vertically?
- Deeply weathered?
- High *in situ* permeability, vertical and horizontal?
- Low *in situ* density at depth?
- If sinkholes present, their mechanism and effect?
- Fine soils suitable for earth core?
- Gravelly ferricrete or alcrete suitable for pavements?
- Cemented material in crust suitable for rockfill or rip-rap?
- Silcrete horizon or quartzite bed?

## 3.12 GLACIAL DEPOSITS AND LANDFORMS

During the Pleistocene period, large parts of the earth's surface were covered by sheets of ice, similar to those which occur today in Greenland, Antarctica, parts of Europe and North and South America. The ice moved across the landscape, eroding and reshaping it, and when it melted it deposited the eroded materials. Similar "ice ages" occurred earlier in the earth's history, but with rare exceptions glacial deposits formed at those times have been so modified and strengthened by diagenesis or metamorphism that they are now rocks not greatly different in engineering properties from the surfaces on which they were deposited. The discussion here is therefore limited to effects of the Pleistocene glaciation and of valley glaciers such as still occur commonly in alpine regions (Figure 3.41).

Gaciated landscapes usually have complex histories of erosion and deposition, including for example, the following sequence shown on Figure 3.42.

Figure 3.41. Aerial view of the Tasman Glacier in New Zealand. Photo courtesy of Mr. Lloyd Homer, DSIR, New Zealand.

- Erosion and shaping by rivers (Figure 3.42a);
- Erosion and reshaping by ice, and deposition of glacially derived materials (Figure 3.42b);
- Erosion and reshaping of the new landscape and the glacial deposits, by subsequent rivers (Figure 3.42c).

Such histories have resulted in a wide variety of landscapes and deposits. The deposits vary widely in their engineering properties. Unfortunately, from examination of the mineral content and texture of a soil it is frequently not even possible to determine whether or not it is of glacial origin. If these characteristics are considered in relation to the landforms on or in which the soil occurs it is often possible to confirm a glacial origin and to make more detailed predictions about its history and thus about its likely distribution and engineering properties.

Boulton and Paul (1976) introduced the "land system" form of terrain evaluation as a means of classifying and mapping of sediment sequences and landforms. This land system approach is used by Eyles (1985) who recognizes three main land systems resulting from glaciation.

- Subglacial and supraglacial – characteristic of lowlands where sediments and landforms were formed by large sheets;
- Glaciated valley – characteristic of areas of high relief in which the ice was restricted to valleys.

Other glacially-related environments are:

- Periglacial – in which intense frost action modified the glacially developed landforms and materials;



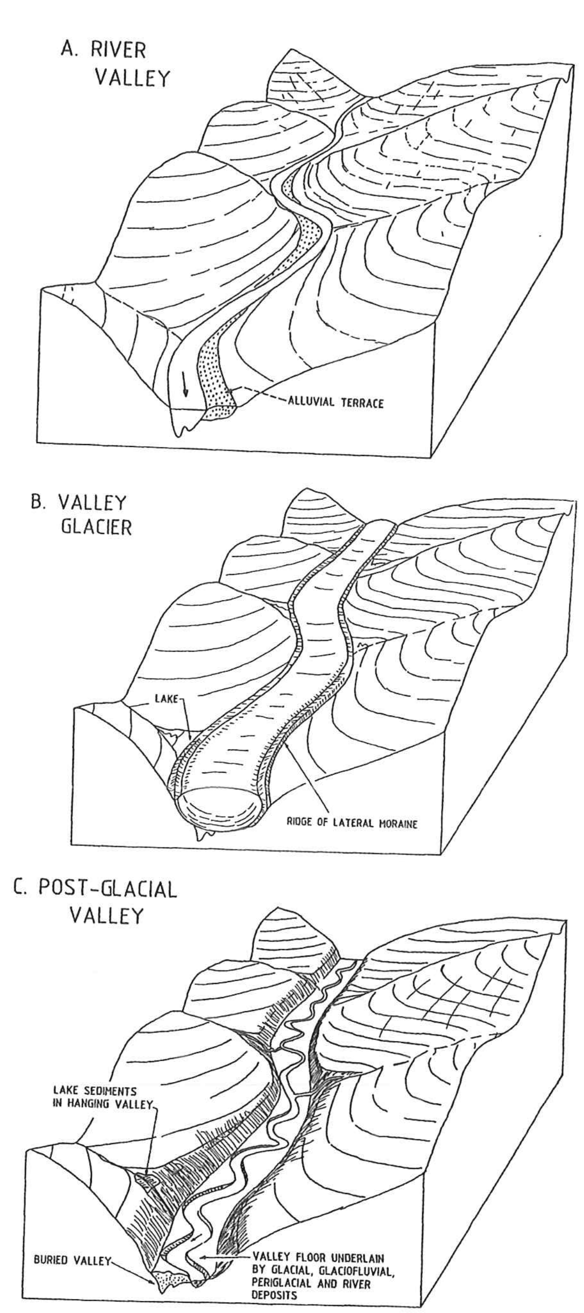



Figure 3.42. River valley, before during and after glaciation.

- Glaciofluvial – in which glacially derived sediments were transported and deposited by water.

The processes and products of each land system and environment are discussed in some detail in Eyles (1985). Discussion in this present chapter will be limited to the glaciated valley land system (the most significant in dam engineering) plus brief notes on glaciofluvial and periglacial aspects.

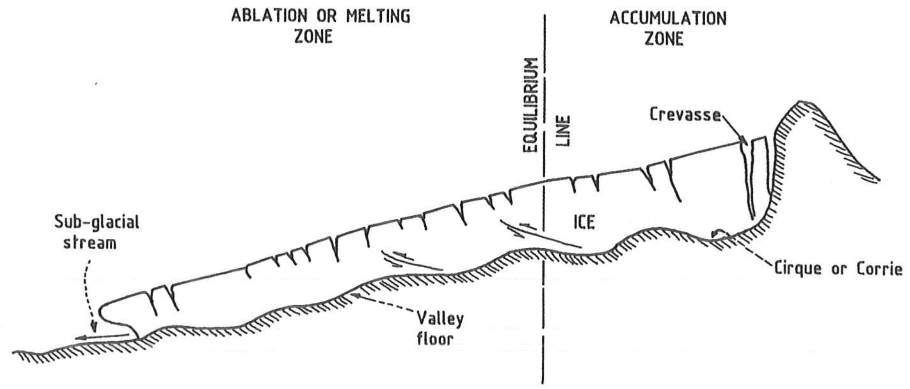



Figure 3.43. Diagrammatic section along a valley glacier (based on Blyth and de Freitas, 1989).

### 3.12.1 *Glaciated valleys*

Figure 3.43 is a diagrammatic section along a valley glacier during a period of its "advance" down the valley. It can be seen that the ice tends to move over irregularities in its path, behaving in a viscous manner near its base and failing in shear and tension at higher levels. The resulting valley floor is uneven and often contains deep hollows eroded by the ice. Many such hollows are now occupied by lakes, the water is dammed partly by ridges of rock and partly by glacial debris. Streams of meltwater flow beneath the ice and exit at the snout.

During periods in which winter snows do not survive through summer the glacier diminishes in size and its snout "retreats" up the valley.

Although glacial valleys are generally U-shaped when viewed broadly, in detail the valley floors are often highly irregular as at Parangana Dam in Tasmania (Figures 3.44 and 3.45). The shape of the buried valleys at this site and nature of the infill materials, indicates that they are remnants of pre-glacial river valleys. Figure 3.44 illustrates the way in which such a pre-glacial river channel can be preserved under glacially derived deposits.

In other cases locally deeper valley sections under glaciers may have been differentially eroded by ice or meltwater streams.

### 3.12.2 *Materials deposited by glaciers*

The general generic term for material deposited by glaciers is "till". Deposits of till are often referred to as "moraine". A further term "drift" has been used extensively and loosely to describe surficial deposits of glacial, alluvial or colluvial origins.

Figure 3.46 shows mechanisms of accumulation, ingestion and transport of debris by a glacier. Upslope from the equilibrium line (Figure 3.46a) windblown dust and rock debris is buried within the snow which feeds the glacier and is transported downglacier forming the basal debris zone and thin basal traction layer. The rock particles in the traction layer abrade, polish and groove the rock floor, generating fine rock particles known as rock flour. The ice and rock blocks within it also pluck or "quarry" rock from the floor and sides. The resulting material developed in contact with the rock floor is known as lodgement till (Figure 3.46b) and usually contains a wide range of particle sizes. The lodgement till is formed under relatively high effective normal pressures and consequently is usually compacted to at least stiff consistency. Its upper surface is grooved or fluted parallel to the direction of flow of the ice.



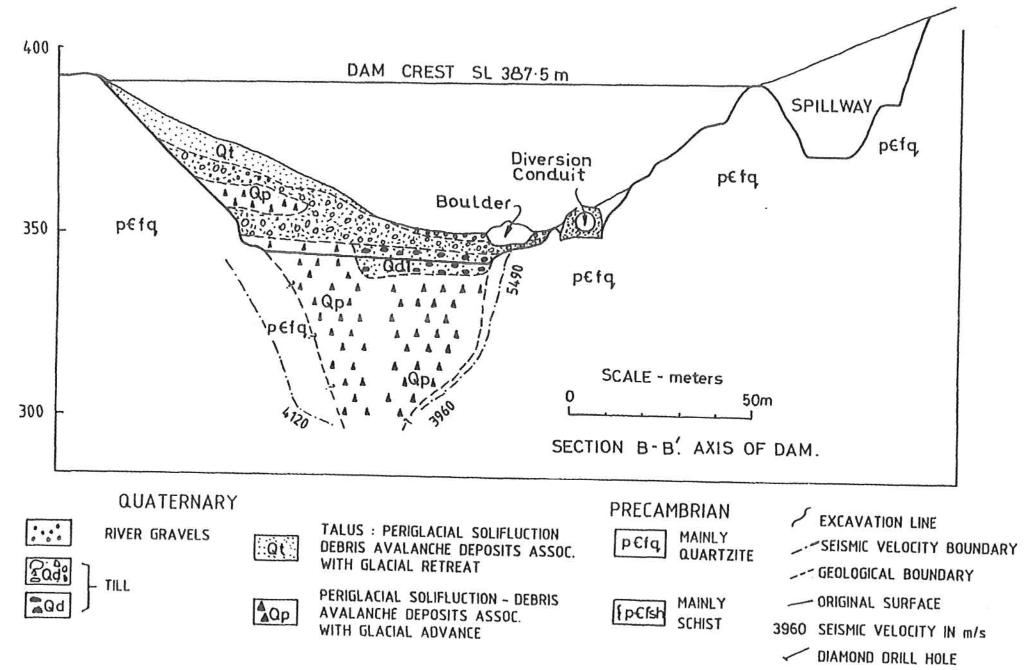



Figure 3.44. Parangana Dam, Tasmania, cross section along dam axis (Paterson, 1971).

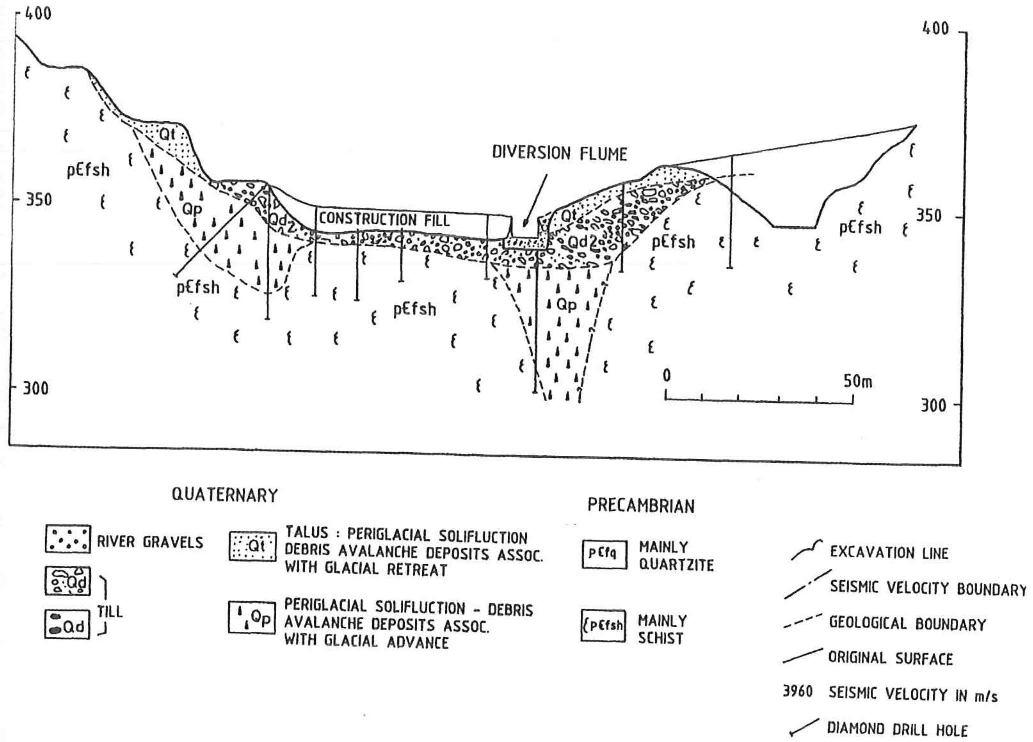



Figure 3.45. Parangana Dam, Tasmania, cross-section 100 m downstream from dam axis (Paterson, 1971).

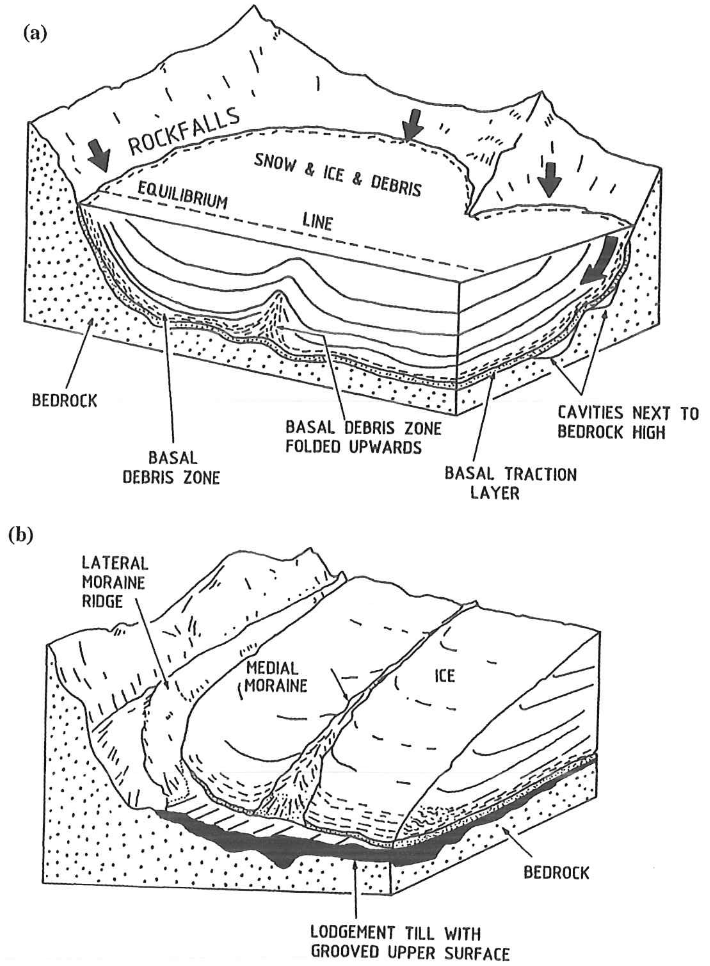



Figure 3.46. Debris transport by a valley glacier (Eyles 1985, by permission of Pergamon Press).

Where two glaciers converge to form a composite valley glacier, the debris zone may be folded upwards by the compression generated along the glacier contacts and in the ablation zone (Figure 3.46b) the debris may become exposed as a supraglacial medial moraine.

Below the equilibrium line (Figure 3.46b) debris falling onto the glacier surface is not ingested by the glacier because the winter snows do not survive the next summer. The debris is transported along the glacier sides as ridges of supraglacial lateral moraine or as supraglacial medial moraine after the convergence of two glaciers.

Figure 3.47 shows a typical situation at the snout where debris from the glacier surface and within it are deposited as the ice melts.

Figures 3.48 and 3.49 show the development of a ridge of supraglacial lateral moraine and soils associated with it during a single advance/retreat stage of a glacier.

During the advance (Figures 3.48a to c), debris which slides off the glacier surface forms steeply dipping deposits (lateral moraine) analogous to talus or end-dumped fill. The material, more precisely known as supraglacial morainic till, includes a wide range of particle sizes. Outwash materials with initial near-horizontal bedding are deposited by streams in the trough between the lateral moraine and the valley side. At depth the accumulated moraine



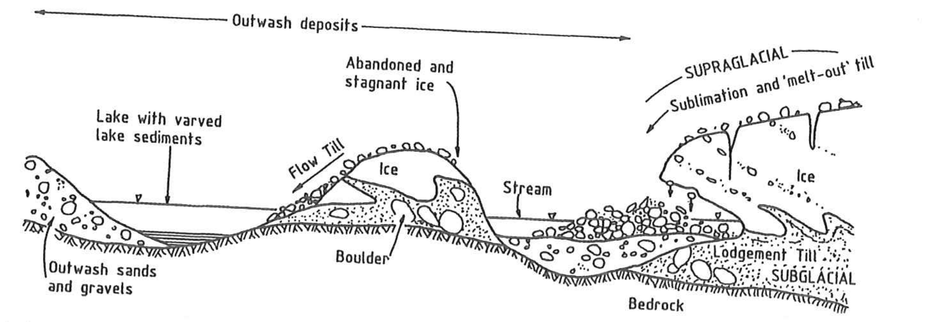



Figure 3.47. Diagrammatic longitudinal section at a glacier snout, and downstream (Blyth and de Freitas, 1989).

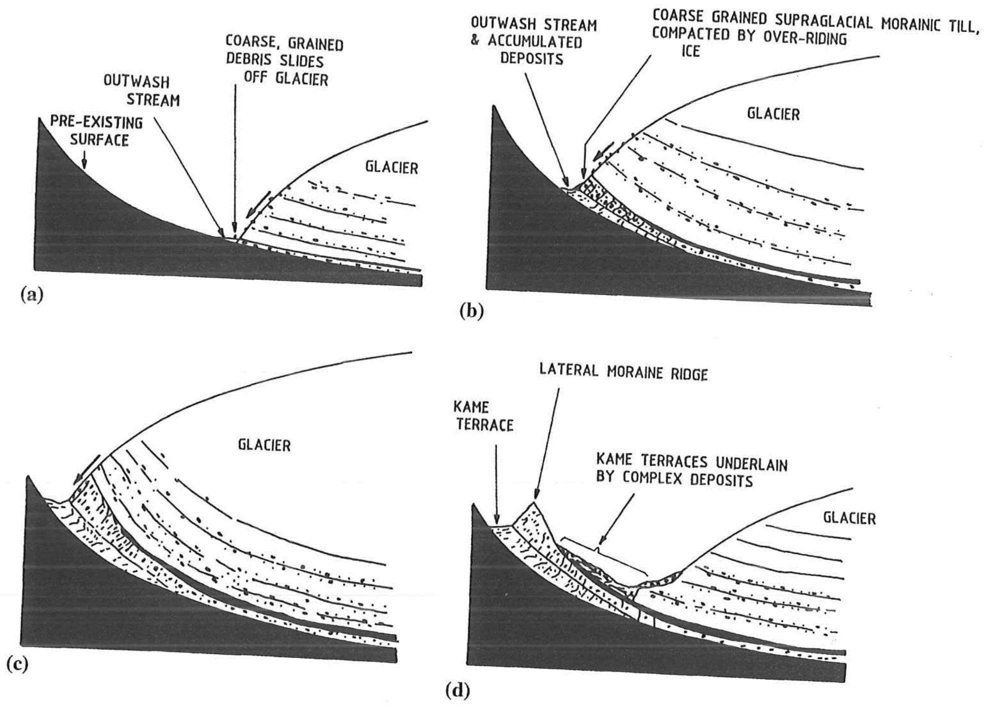



Figure 3.48. Development of a lateral moraine ridge and associated deposits (Boulton and Eyles, 1979).

and outwash materials are compressed and compacted by the glacier and their dips become steeper.

During the retreat (Figure 3.48d) the glacier shrinks away leaving the lateral moraine as a ridge. Such ridges often remain steep (up to 70°) due to the high degree of compaction and some cementation of the moraine. Between the ridge and the ice a series of terraces develops. These are known as kame terraces and are underlain (Figures 3.48d and 3.49) by complex sequences including glaciofluvial sands and gravels (stream-deposits), laminated clays (lake-deposits), poorly- or non-compacted till (supraglacial morainic till) and lodgement till.

Terraces formed by the outwash next to the valley side are also known as kame terraces (Figure 3.49).

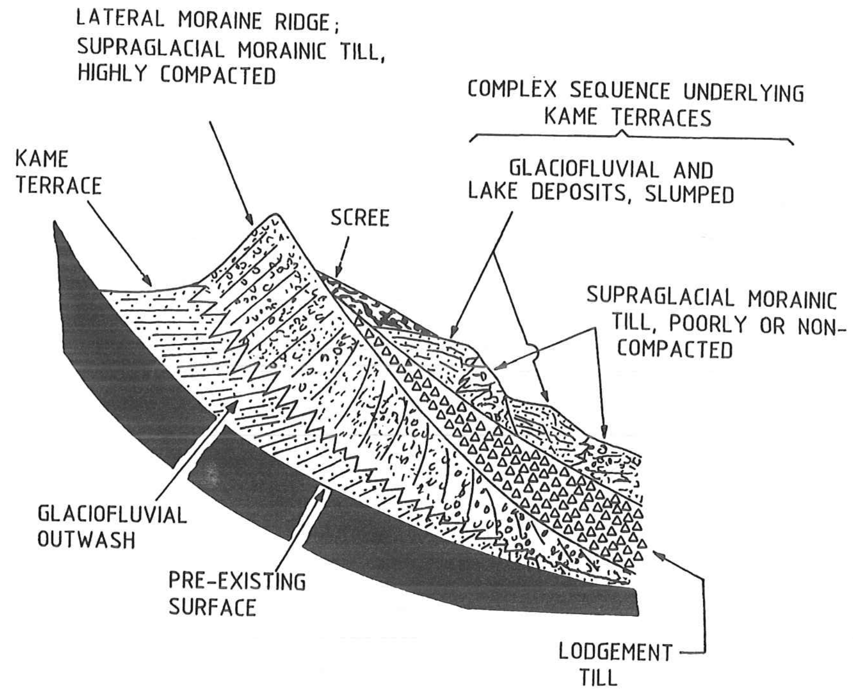



Figure 3.49. Diagrammatic section through a lateral moraine ridge (Boulton and Eyles 1979).

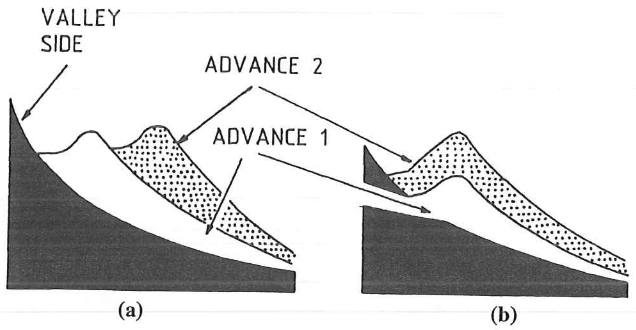



Figure 3.50. Lateral moraine ridges deposited by successive glacial advances. (Boulton and Eyles, 1979).

The lateral moraine and associated deposits are usually more complex than shown on Figure 3.49 because of repeated glacial advances and retreats. Figure 3.50 shows the relationships between lateral moraine ridges deposited by successive glacial advances. In Diagram (a) Advance 1 was more extensive than Advance 2, and in Diagram (b), Advance 1 was the less extensive.

Figure 3.51 is the complete glaciated valley land system, showing common features of the ablation zone and the deposited materials and landforms. Numbered features on this diagram which need further explanation are discussed below.

The lodgement till (1) at the base of the glacier is poorly sorted, usually containing more than 50% of sand, silt and clay sizes, forming a matrix which supports gravel and larger sized particles. When unweathered, the fines fraction usually comprises finely ground quartz, carbonate and inactive clay minerals. The lodgement till is well compacted



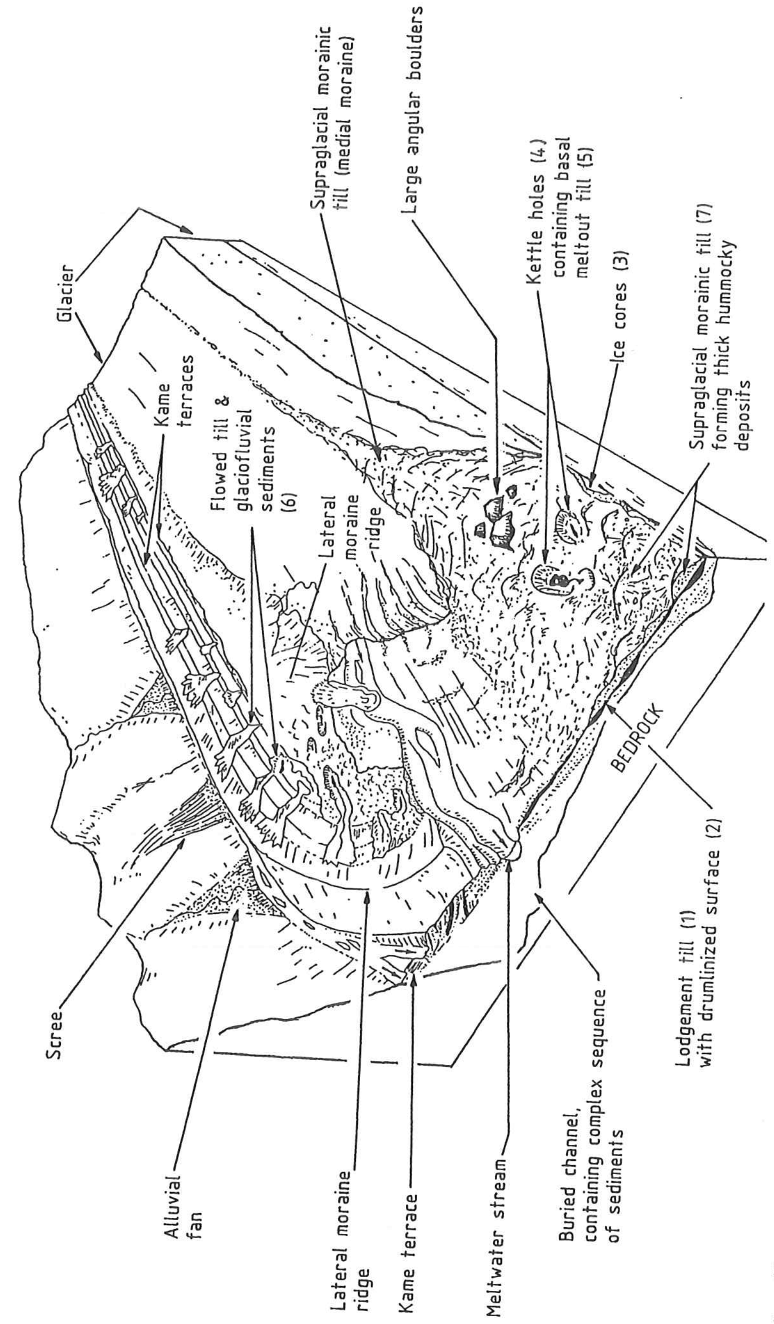



Figure 3.51. Landforms and sediments associated with the retreat of a valley glacier (based on Eyles 1985, by permission of Pergamon Press).

by the overlying ice, has a well developed fissured fabric parallel to the ice flow direction and is commonly highly deformed (Figure 3.46). It usually has very low permeability, but may be rendered locally permeable in the mass by thin glaciofluvial sand layers deposited in channels of meltwater streams.

The till near the downstream toe at Parangana Dam (Figure 3.45) ranged from "toughly compacted' to weak rock and is considered by Paterson (1971) to be lodgement till.

Drumlins (2) are elongated dune-like mounds on the surface of the lodgement till, apparently moulded to this shape by the moving ice.

Ice-cores (3) are masses of ice left behind by a retreating glacier and buried in till. Melting of ice-cores causes the development of sinkhole-like features known as kettles (4) and deposition of basal melt-out till (5) which is usually crudely stratified.

Flowed till (6) is till which has been reworked and deposited by mudflows, the scars of which can be seen extending from the lateral moraine ridge through its adjacent kame terraces. The flowed till may be stratified due to redistribution of fines during flow. The kame terraces here are underlain by complexly interbedded sediments as shown on Figure 3.49.

Supraglacial morainic till (7) deposited at the snout, as shown on Figure 3.47, contains a large range of gravel and larger fragment sizes and is usually deficient in fines. It is known also as supraglacial melt-out till or ablation till. The younger glacial deposit at the dam axis area at Parangana Dam (Figure 3.44) is believed to be of this type. It comprises up to 70% of gravel to boulder sizes in a matrix of sand, clay and silt. The largest boulders are up to 3 m diameter.

#### 3.12.2.1 *Properties of till materials*

It is clear that basal melt-out till, flowed till and supraglacial morainic till will be poorly consolidated unless they become desiccated or covered by new sediment or ice at a later time in their history. The latter appears to be the case at Parangana Dam where the supraglacial morainic till is well compacted, apparently due to being overridden by ice during subsequent glacier advances (Paterson, 1971). Experience has shown that most tills of all types have low permeabilities (less than $10^{-10}$ m/s). However, till deposits may contain or be next to bodies of permeable sands and/or gravels of glaciofluvial origin, filling old channels. For example the various outwash deposits shown on Figure 3.48 would normally have relatively high permeabilities. At Parangana Dam the younger till beneath the valley floor contained a layer of glaciofluvial sands several metres thick with measured permeability of $10^{-6}$ m/s. To allow for this layer the core cutoff trench was excavated down into underlying materials of low permeability (Figure 3.44). The younger till was left in place beneath the downstream shoulder, but was covered with a filter blanket before placement of the rockfill.

Walberg et al. (1985) describe remedial works required at the 23 m high Smithfield Dam in Missouri, USA, after seepages and high piezometric pressures were recorded during first filling. Exploratory drilling using 152 mm diameter cable tool tube sampling showed that glacial outwash sands and gravels beneath the left abutment were more continuous and permeable than assumed from the pre-construction drilling and sampling.

At Cow Green Dam in Britain most of the material filling a buried channel beneath the left side of the valley was found to be lodgement till, described as "stiff, dark brown, poorly-sorted, unstratified, silty, sandy clay of medium plasticity containing subangular to rounded gravel, cobbles and boulders.....". The boulders ranged up to 2 m in mean diameter (Money, 1985).

Sladen and Wrigley (1985) describe further generalisations which can be made about the geotechnical properties of lodgement tills.

#### 3.12.2.2 *Disrupted bedrock surface beneath glaciers*

Knill (1968) describes the open-fractured nature of bedrock beneath glacial materials at several sites and concludes that gaping or infilled joints near-parallel to the rock surface



were initiated as shear fractures by the moving ice (i.e. by "glacitectonic thrusting") and then opened up by ice wedging. The authors have found similarly fractured rock with open and infilled joints at the bases of many valleys which have not been subjected to glaciation (see Chapter 2, Section 2.5.4) and suggest that the effects seen by Knill may have been formed largely by stress relief.

Regardless of their origin, the presence of such features at many sites means that the rock next to the base of glacial deposits is likely to be of poor quality e.g. in terms of compressibility, permeability and erodibility. Where the existence of such poor quality rock would be of significance to the stability and/or watertightness of the dam it is important that this "rock-head" zone be investigated thoroughly.

Money (1985) draws attention to the difficulties often encountered while doing this by core drilling and mentions cases where large boulders have been mistaken for bedrock. He refers to UK practice at that time which was to recommend that the *in situ* rock be proven by a minimum of 3 m of cored rock. He states that this figure is likely to be inadequate and that it is certainly not enough to allow for adequate permeability testing of the upper part of the bedrock. The authors agree and suggest that the actual depth of coring needed will depend upon:

- The inherent fabric of the bedrock (i.e. is it massive or too well-cleaved or closely jointed for it to have formed large boulders?);
- The quality of the core samples and the extent to which core orientation can assist in assessing whether it is *in situ* or not. Core orientation may be determined either by impression packer or orientation device or by the presence of bedding or foliation with known, consistent orientation;
- The actual depth of the disturbed zone.

Difficulties in delineation of the top of *in situ* rock can occur also where the rock in this zone has been chemically weathered. This situation exists at the site for Kosciusko Dam in New South Wales (Figure 3.52).

It appears that intense weathering has occurred in both the bedrock and the till. Geological surface mapping, track exposures and boreholes on the right bank show that most of the upper 5–20 m of the bedrock comprises residual "boulders" of fresh to slightly weathered granite set in a matrix of highly to extremely weathered granite which is mainly a very compact silty, clayey sand.

A shaft close to the creek on the left bank (Figure 3.53) showed similarly weathered granite boulders set partly in a matrix of gravelly clay (till) and partly in glaciofluvial sands and clays. It was clear that without very good recovery of little-disturbed core, these materials could not be readily distinguished from the *in situ* weathered "bouldery" sequence. At 8.6 m these materials rested on extremely weathered rock whose mineral content and foliation attitude matched the known bedrock on the right bank. This weathered rock was therefore inferred to be *in situ*.

Holes drilled elsewhere on the left bank recovered about 20% of fresh or slightly weathered granite in "boulder" lengths, but little or no matrix. Hence the upper surface of the *in situ* rock (assuming that it was more than slightly weathered) could not be determined here from the drilling results. The top of mainly fresh granite was inferred from the drill cores together with the results of refraction seismic traverses (Figure 3.52).

Glastonbury (2002), Glastonbury and Fell (2002c) note that many of the large rapid landslides which have occurred in historic times have occurred in valleys which have been glaciated, and attribute this to the stress relief effects on the valley sides as the glaciers retreat.

### 3.12.3 *Glaciofluvial deposits*

As well as depositing some glaciofluvial materials in their immediate vicinity (as discussed in Section 3.12.2) glaciers release very large meltwater flows giving rise to deposition of

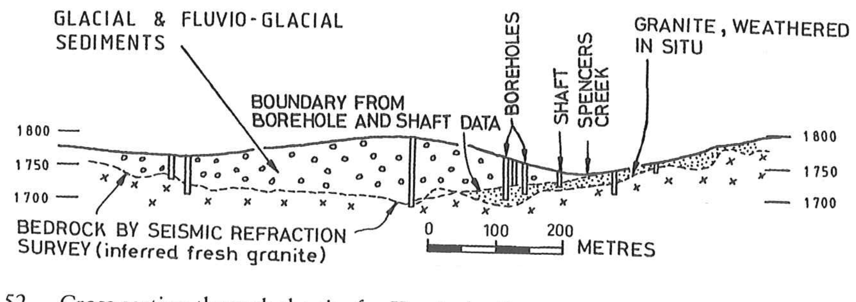



Figure 3.52. Cross section through the site for Kosciusko Dam.

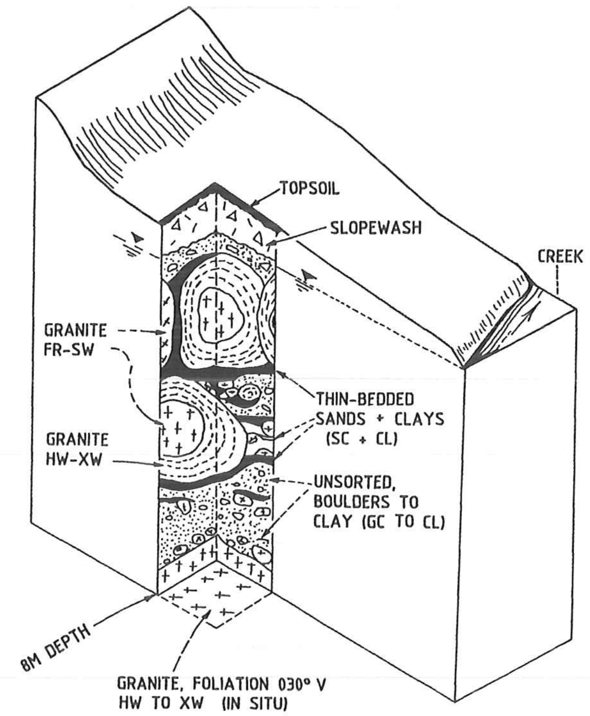



Figure 3.53. Log of shaft, Kosciusko Dam site.

vast amounts of gravels and sands in braided (multiple-channel) rivers usually extending tens of kilometres downstream, often across broad outwash plains (Figure 3.54). Grain sizes range from gravel-dominated near the glaciers to sands and silts further downstream. In some places lakes are formed, in which laminated silts and clays are deposited.

Deposition, eroding and reworking of the braided river deposits occurs cyclically in phase with glacial advances and retreats, resulting in terraces at various levels across the river valleys and outwash plains. The streams eventually flow into lakes or the sea.

Exposed areas of silt and fine sand are eroded by wind and redeposited as loess on the surrounding country.

Miall (1985) describes sedimentological aspects of the braided stream deposits.

The sands and gravels are usually clean with well-rounded particles and often provide excellent sources of materials for embankments and for concrete aggregate.



Figure 3.54. Broad valleys downstream from glaciers in New Zealand, underlain by great depths of glaciofluvial gravels and sands. Photo courtesy of Mr. Lloyd Homer, DSIR, New Zealand.

The braided streams and lakes produce very complex, lenticular deposits of sands, gravels, silts and clays. Terzaghi and Leps (1958) describe the design, construction and performance of Vermilion Dam, a 39 m high zoned earthfill structure built on such complex glaciofluvial deposits, with maximum depth of 82 m.

In some deposits there are lenticular beds of "open-work" gravels or boulders which are uniformly sized materials with large voids and extremely high permeability. Cary (1950) describes the widespread occurrence of these materials in glaciofluvial deposits in northwestern USA.

### 3.12.4 *Periglacial features*

Periglacial conditions are defined here as those under which frost is the predominant weathering agent. They are often, but not always, associated with glaciers. Permafrost conditions are commonly present but are not essential. Permafrost occurs where winter temperatures are rarely above freezing point and summer temperatures are only high enough to thaw the upper metre or so of the ground.

Figure 3.55, modified slightly from a diagram of Eyles and Paul (1985), shows features which may be developed under periglacial conditions.

1. Deep-seated creep of weak sedimentary rocks into the valley. Competent beds develop widely gaping or infilled extension joints (gulls) and move downslope on the weak materials as complex slides or rafts;
2. Weak rocks contorted and bulged upwards, overlain by terraced gravels;
3. Outcrops showing evidence of toppling and cambering, with scree and rockfall deposits downslope;
4. Outcrops of very strong crystalline bedrock surrounded by blockfield of frost-heaved bedrock. Terraces cut by nivation – free-thaw and slopewash at the margins of snow patches;

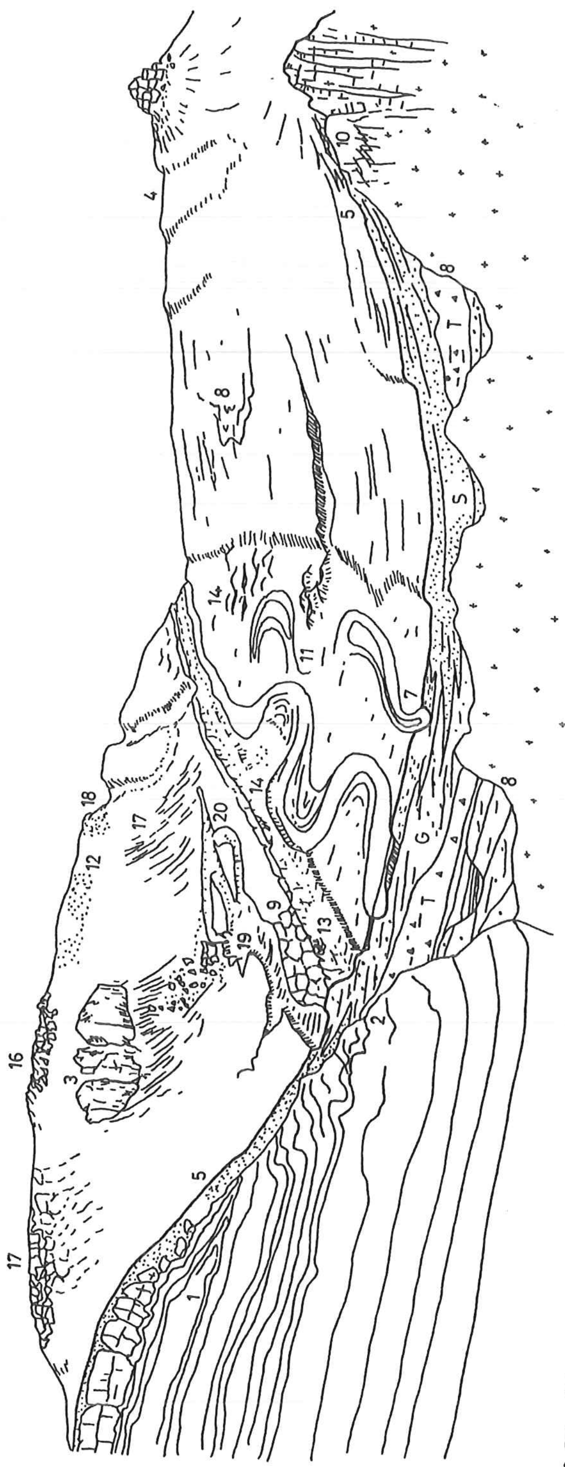


Figure 3.55. Diagram showing features which may be developed under periglacial conditions (Based on Eyles and Paul, 1985).



5. Solifluction fans or lobes – crudely bedded gravelly or bouldery deposits, thickening downslope, grading into slopewash beneath the lower slope;
6. Mudflow scar underlain by low-angle shear surface;
7. Mantle of solifluction debris (S) intertongued with gravels (G) deposited by periglacial braided stream, and overlain by peats and silts (M) forming the floodplain of the modern meandering stream;
8. Till (T) in the buried valley and in pockets in the rockhead;
9. Polygonal patterned ground and fossil casts of ice-wedged cracks;
10. Frost-shattered bedrock;
11. Doughnut-shaped degraded ramparts left by former pingos;
12. Soil mantle resulting from mechanical churning (frost heaving) of pedological soils and bedrock. The underlying rockhead is often highly irregular;
13. Involutions and contortions in the soil caused by high pore pressures developed during re-freezing of thaw-soaked soil;
14. Faulted soil zones caused by ground contraction during freezing and/or collapse into voids following melting of buried ice;
15. Patterned ground – polygonal nets and stripes;
16. Blockfield;
17. Alluvial fans;
18. Dry Valleys;
19. Terraced gravels (G) deposited by braided periglacial streams. The bedrock surface below may contain scoured channels and hollows;
20. Loess.

Eyles and Paul (1985) point out that most of the features indicated on this diagram may also develop (more slowly) in warmer climates. The authors agree and in particular consider it likely that the "Deep-seated disturbances" Features 1 and 2 would have been developed to a large extent as a result of destressing (as described in Chapter 2) before any cold-climate effects.The following explanation of the main cold-climate processes and features indicated on Figure 3.55 is based also on Eyles and Paul (1985).

Solifluction (7) is the slow flowage or creep of a water-soaked mass of soil and rock debris, either as a true flow or as a slide where most movement occurs over a basal shear surface. Typical flow rates range from 10 mm to 60 mm per year. Solifluction is caused by the generation of excess pore pressures during the thaw. Eyles and Paul (1985) provide a summary of theoretical studies of the process by Nixon and Morgenstern (1973) and McRoberts and Morgenstern (1974).

Another solifluction mechanism is the downslope displacement of soil particles by needle-ice. The ice needles grow normally to the ground surface and so during each freeze-thaw cycle the supported particles are displaced slightly downslope.

Frost-shatter (10) is the mechanical disruption of rock masses by the expansion on freezing of groundwaters.

Cyroturbation (12), (13) and (15) is the churning or mixing of soils which occurs in permafrost conditions at the end of the melt season due mainly to high pore pressures set up towards the base of the thawed layer as the soil re-freezes from the surface down. Expansion due to freezing of porewaters also contributes, as does the upward heave of cobbles and boulders resulting from their greater thermal conductivity. The "involutions" (13) are pseudo-intrusive structures similar to flame structures in sediments and to gilgai in clays. Patterned ground (15) is ground showing a regular hummocky pattern (polygons or circles). It is a surface expression of cyroturbation.

Ice-wedging (9) refers to cracks which occur in soils during intense cold widened by wedging action when water freezes in them. The cracks are preserved as casts by soil which migrates into them.

Pingos (11) are conical mounds of buried ice up to 40 m high and 600 m in diameter. Pingos, which have developed by the freezing of upward-moving groundwater, have caused updoming of the surrounding sediments. Melting of the ice results in the doughnut-shaped surface features shown on Figure 3.55.

Paterson (1971) provides a useful account of periglacial and other effects of glaciation features at Parangana Dam. Core-drilling into the steep sided buried pre-glacial river channels at the site (Figures 3.44 and 3.45) showed the lower part of each to be filled mainly by angular rock fragments up to 500 mm across set in a sandy clay to clayey sand matrix. The ratio of rock to matrix was roughly 60 : 40. The rock fragments were of quartzite and schist, clearly derived from the bedrock immediately upslope in each case. These deposits were judged to have been formed by "solifluction debris avalanches" from the steep valley sides, under periglacial conditions during the glacial advance period.

In the upstream channel the avalanche material contained a discontinuous bed of clay believed to have been deposited in a lake formed upstream from one of the slides. Also present in the slide deposits were lenticular beds of sand and gravel, inferred by Paterson to be stream deposits in channels eroded through the slide dams. The distribution of sand and gravel beds suggested that at least five major landslides had occurred.

Locally derived talus covers most of the ground surface at the site and extends to about 10 m depth in the foundation area (Figures 3.44c and 3.45). This material is believed to be a periglacial solifluction product formed during the final retreat stage of the glacier.

### 3.12.5 *Glacial environment – list of questions*

Only the most significant glacier-related questions are listed here; these and other features which are usually of less significance in dam engineering are shown on Figures 3.51 and 3.55.

- Buried valleys?
- Bedrock surface or boulder?
- Bedrock disrupted near upper surface?
- Wide variety of till types?
- Materials unsorted – clay to boulder sizes?
- Slickensides in clay-rich till?
- Variable compaction and cementation?
- High permeability sands and gravels?
- Loess?
- Landslipped deposits?
- Creeping landslides?

# 4

# Planning, conducting and reporting of geotechnical investigations

## 4.1 THE NEED TO ASK THE RIGHT QUESTIONS

The French detective, Bertillon, considered by many to the father of modern crime detection, is reputed to have made the following statement.

*"We only see what we observe, but we can only observe that which is already in the mind"*. Experience from analysis of many case histories (Stapledon, 1976, 1979 and 1983) shows that this principle (that we will only find that which we recognise) is equally true in engineering site investigations. In almost every foundation failure and contractual dispute over "changed geological conditions", it is found that a major contributing factor has been the failure of project planners and site investigators to fully understand and define all of the geotechnical questions which needed to be answered by the site investigations. There are two types of questions, namely:

- *engineering questions*, which relate essentially to the design, construction and operation of any structure of the type proposed, and
- *geological questions*, which arise from understanding of the site geological environment and its likely influence on the design, construction and operation of the project (see Chapters 2 and 3).

### 4.1.1 *Geotechnical engineering questions*

For dams which are intended to store water, or water plus solids, it is obvious that important questions must relate to the permeability of the foundations. However, there are many other equally important questions, because construction and operation of a dam causes much greater changes to a site environment than any other type of engineering activity. Four main processes involved in dam engineering, their principal effects on the site environment, and some resulting questions for the designer and site investigator, are as follows;

1. Excavation: To reach suitable levels for founding the dam, and also for the spillway, outlet works and for construction materials. Excavation causes removal of support from and increases in shear stress in the surrounding material and hence raises questions of stability of the excavations themselves, during construction and/or operation, and of the stability of the adjacent valley slopes.
2. Foundation loading: Imposed by the dam structure, raises questions of compressibility of the foundation and its shear strength against sliding upstream or downstream, before and after filling of the storage and under flood and earthquake loading.
3. Inundation – Filling the storage: Causes changes to the groundwater regime, lowering of strengths of cohesive soils, weak rocks and joint cements, and decreases in effective stress. These effects all add to the questions of stability of the dam and its foundation and also they raise the question of stability of the reservoir sides. These stability questions are more serious when water storage levels are required to fluctuate widely and rapidly.