GEOTECHNICAL ENGINEERING OF EMBANKMENT DAMS

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Preface

This book sets out to present a comprehensive coverage of geotechnical engineering of embankment dams. It is based on the authors' experience in dam engineering in Australia, Asia and the Pacific Islands, but also draws on international literature to present an overview of practice throughout the world.

The engineering of embankment dams requires a detailed understanding of the influence of geology on dam foundations and construction materials, as it is usually necessary to use what nature has made available, rather than to seek ideal conditions.

Practitioners, therefore, need to know what to expect in any geological environment, and to have an understanding of the limitations of investigation and design methods, and construction procedures. The authors have set out to provide this knowledge and understanding.

The authors wish to acknowledge the considerable assistance given by the many engineers and geologists with whom they have worked, and who have provided information for the book. They also wish to thank Gwenda Taylor, University of New South Wales, and Dawn Leonard, University of South Australia, who prepared the manuscript, and Dr. G. Tamaddoni, University of New South Wales who proof read the text.

CHAPTER 1

Embankment dams, their zoning and selection

1.1 TYPES OF EMBANKMENT DAMS, THEIR ADVANTAGES AND LIMITATIONS

There are several types of embankment dam. The designs have varying degrees of in-built conservatism, usually relating to the degree to which seepage within the dam is controlled by provision of filters and drains, the use of free draining rockfill in the embankment, and the control of foundation seepage by grouting, drainage and cutoff construction.

1.1.1 *Dam failures causes and frequency*

An ICOLD (1973) survey of the causes of dam failures reproduced in Schnitter (1979) and in National Research Council (1983) showed that apart from over-topping, piping and seepage failure were the main reason for dam failure. (Fig. 1.1).

This study, which only applied to dams of height greater than 15 m, also showed that:

– foundation failures usually occur relatively early in the dam life,

– 50% of all the failed dams were between 15 and 20 m high,

– earth embankment dams account for 74% of all dam failures, and historically were nearly twice as likely to fail as concrete dams. However, after about 1985, the probability of failure of embankment dams was similar to that of concrete dams.

A USCOLD survey of dam incidents up to 1979, reported in National Research Council (1983) is summarised in Table 1.1. Again, it can be seen that leakage and piping through the dam embankment and foundation are the primary causes of failure and 'accidents.' It is also important to control seepage pore pressures in embankment dams, since the stability of the embankment is dependent on these pore pressures. In the design phase in particular, one must be able to confidently predict the pore pressures. This prediction becomes a significant factor in the selection of embankment zoning.

1.1.2 *Consequences of failure, or hazard*

As discussed in Chapter 18 dams are commonly rated as to their hazard, i.e. to the damage and loss of life which would result following (hypothetical) failure of the dam. This rating may change throughout the life of the dam, e.g. if new development takes place downstream.

Because some embankment types have more in-built conservatism than others, the hazard rating at any particular site becomes an influencing factor in the choice of embankment type best suited to that site.

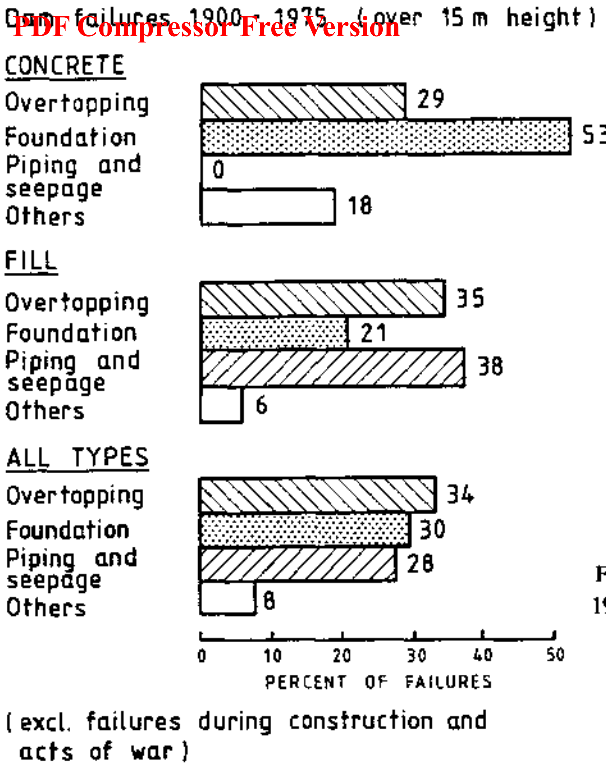


Figure 1.1. Causes of dam failure (ICOLD 1973 and National Research Council 1983).

Table 1.1. Causes of dam incidents.

	Type of dam								
	Concrete		Embankment		Other*		Totals		
Cause	F	A	F	A	F	A	F	A	F&A
Overtopping	6	3	18	7	3		27	10	37
Flow erosion	3		14	17			17	17	34
Slope protection damage				13				13	13
Embankment leakage, piping			23	14			23	14	37
Foundation leakage, piping	5	6	11	43	1		17	49	66
Sliding	2	5	28				7	28	35
Deformation		2	3	29	3		6	31	37
Deterioration		6	2	3			2	9	11
Earthquake instability				3				3	3
Faulty construction	2			3			2	3	5
Gate failures	1	2	1	3			2	5	7
TOTAL	19	19	77	163	7		103	182	285

*Steel, masonry-wood, or timber crib.
F = failure.
A = accident = an incident where failure was prevented by remedial work or operating procedures, such as drawing down the pool.
Source: Compiled from Lessons from Dam Incidents, USA, ASCE/USCOLD 1975, and supplementary survey data supplied by USCOLD.

1.1.3 *Dam types*

Figures 1.2 and 1.3 show simplified sections through the various types of embankment dams. Greater detail for several of the dams is given in Section 1.2. The typical features, advantages and limitations of each dam type are as follows.

a) Homogeneous earthfill. Usually constructed from clay, sandy clay, clayey sand and gravel-sand-clay soils but may be constructed from more permeable soils such as silty sand, sand, and sandy gravel provided seepage is acceptable. Provides no internal seepage or erosion control, and so is susceptible to piping failure and seepage emerging on the downstream face. Homogeneous earthfill has been used for larger dams in the past, but should be limited to low (say less than 5 m height dams), in low hazard locations.

b) Earthfill with toe drain. Constructed from similar materials to homogeneous dam, with a permeable rock or gravel toe. Provided the ratio of horizontal to vertical permeability for the earthfill (kH/kV) is near unity, the toe will attract the seepage. With an adequate filter between the earth and rockfill, internal erosion and seepage will be controlled. However kH/kV is commonly greater than unity, with values of 9 or more not unusual in rolled earthfill. In this case seepage may emerge in an uncontrolled manner on the downstream face, giving poor control of internal erosion and seepage. Earthfill with toe drain has been used for larger dams in the past but should be limited to low (say less than 10 m) dams in low hazard locations.

c) Zoned earthfill. Constructed with an earthfill upstream or central core of clay, sandy clay, clayey sand and gravel-sand-clay soils. The outer zones of earthfill are similar materials but compacted to a lesser degree of compaction and/or at a lower water content than the core, so the outer zone is higher permeability than the core. Alternatively the outer zone may be constructed from weathered or low strength rock which will break down under compaction to form a soil/rock mixture, with permeability greater than that of the core, but with sufficient finer grained soil to provide some degree of internal erosion control (without necessarily meeting normal filter design criteria).This provides good internal seepage control provided that there are no concentrated flows through the core (e.g. if cracking occurs), however, internal erosion is not totally controlled since the outer zone is not designed as a filter to the earthfill core. Zoned earthfill has been used for dams greater than 30 m high in the past (for examples see Sherard et al (1963). Not commonly used for dams higher than about 20 m, or for dams in other than low to medium hazard sites.

d) Earthfill with horizontal drain. Constructed from similar materials to a homogeneous dam but with a horizontal drain in the downstream part of the dam composed of high permeability sand or sand and gravel. Provided kH/kV for the earthfill is near unity, the horizontal drain is effective in drawing the seepage flow from the downstream slope. In this case if the drain is designed to act as a filter, internal erosion is also controlled however, as for the case of an embankment with a toe drain, kH/kV is often quite high, and seepage may bypass the horizontal drain and emerge on the downstream slope. In this case neither internal seepage or erosion are well controlled. In the past dams up to 50 m high have been constructed with only a horizontal drain (i.e. no vertical drain), (see Sherard et al. 1963). While such existing dams may be considered adequate for continuing operation, provided they are instrumented and monitored, it is not currently considered good practice. Earthfill embankments with horizontal drains only, should be limited to low (say less than 10 m) dams with a low or medium hazard rating.

e) Earthfill with vertical and horizontal drains. Constructed from similar materials to a homogeneous dam, but with a vertical drain (also known as a chimney drain) and a horizontal drain composed of high permeability sand, or sand and gravel. The vertical drain intercepts

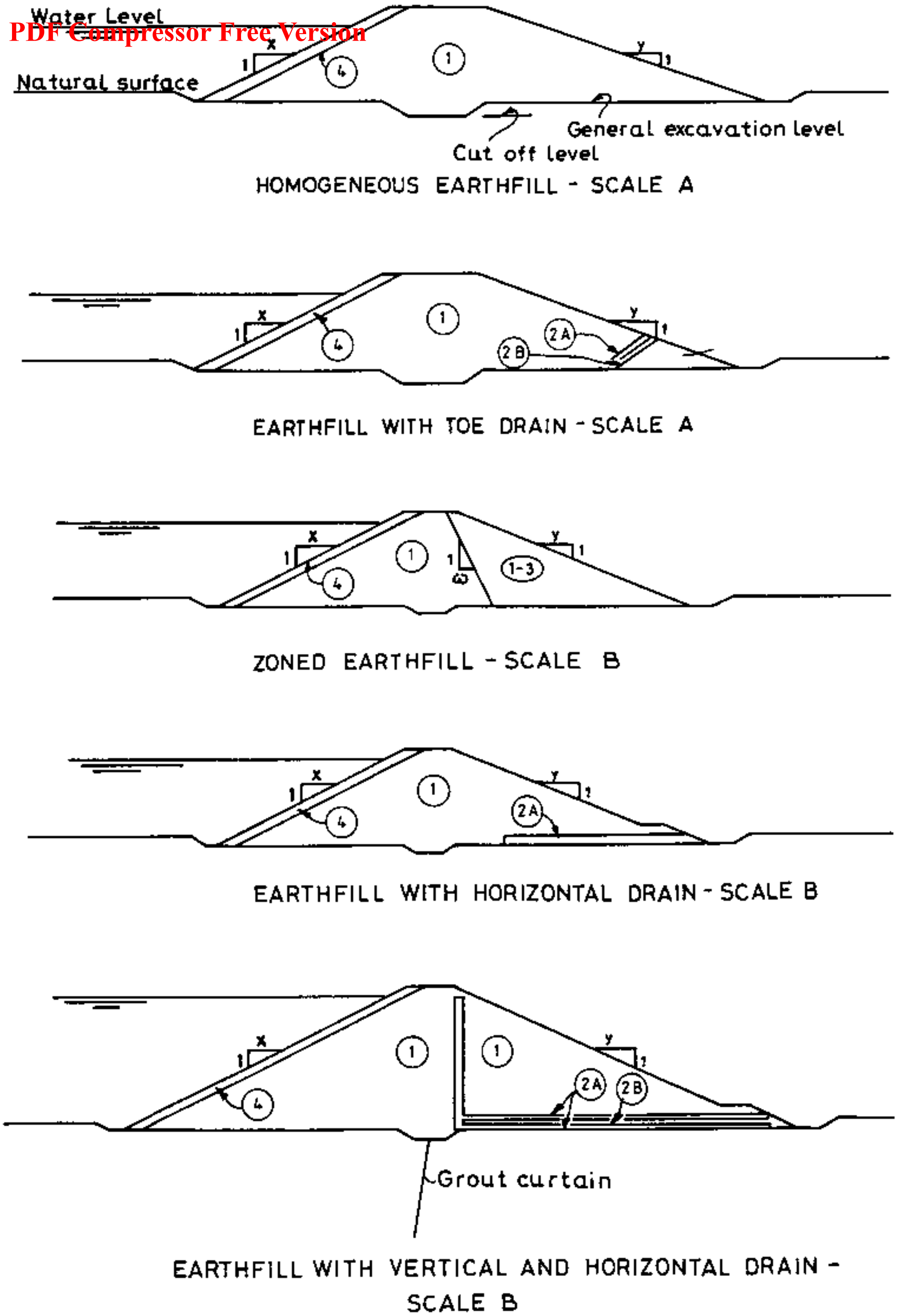


Figure 1.2. Types of embankment Dams.

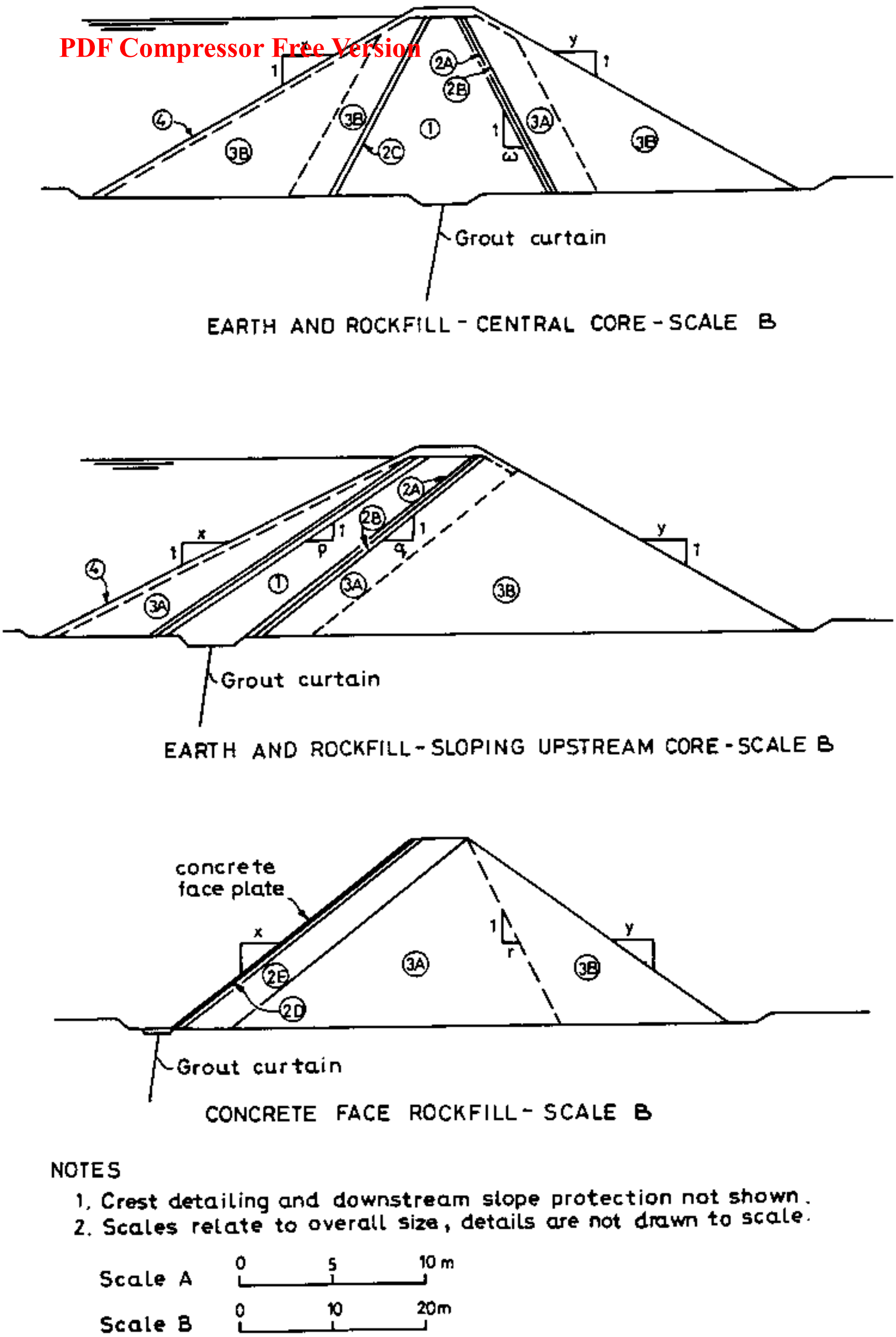


Figure 1.3. Details of earth and rockfill and concrete face rockfill dams.

seepage through the dam and, provided this drain and the horizontal drain have sufficient flow capacity, the earthfill downstream of the vertical drain will remain unsaturated. This control is independent of the kH/kV ratio for the earthfill. If the drains are designed to act as filters to the earthfill, internal erosion is also controlled. The horizontal drain also acts to intercept seepage through the dam foundation and control pore pressures in the embankment due to the underseepage. If the drain is designed as a filter to the foundation material it will also control erosion of the foundation. This is particularly important for dams constructed on permeable soils (sand, sand-gravel and clayey soils with permeable structure); or on weathered permeable rock which is potentially erodible. This type of dam is suited for construction of large dams and there is no reason in principle why even the highest dam in high hazard sites cannot be constructed in this manner. There are many examples of such dams 30 to 50 m high.

f) Earth and rockfill, central core. The central core is constructed from earthfill as described for homogeneous dams, but may be constructed of higher permeability materials such as silty sand or well compacted weathered rock provided that the permeability (and resultant seepage) is acceptable. The core is flanked by rockfill zones upstream and downstream. Ideally the rock used is strong and durable, to provide rockfill which is permanently free draining. In practice, weaker rocks are often used. Filter zones are provided between the core and the rockfill to control internal erosion. Provided that the rockfill is sufficiently permeable (as is usually the case) seepage through the earthfill core is readily discharged and the downstream rockfill remains essentially dry. The pore pressures in the earthfill core are dependent on kH/kV but as the earthfill is supported by the rockfill, the stability is not particularly sensitive to these pressures. Central core earth and rockfill dams are suited for construction of the largest dams. Dams up to 300 m high have been constructed in this manner. For dams less than about 15 to 20 m high the restricted space makes construction complicated and other design types may be more economic.

g) Earth and rockfill, sloping upstream core . Constructed of the same materials as central core earth and rockfill dams. The core is located towards the upstream face of the dam. This may have advantages in some sites as:

– somewhat less earthfill may be required than for a central core dam;

– the core material and filters may be placed after the downstream rockfill, allowing rockfill construction to proceed in wet weather when placement of earthfill may be impracticable;

– staged construction is facilitated by positioning the core at or near the upstream slope;

– the downstream slope of the dam may be steepened but this will usually be more than balanced by the need to provide a flatter upstream slope than required for a central core dam.

Control of seepage and internal erosion is good. As for central core earth and rockfill construction sloping upstream core earth and rockfill dams have been used for many large dams and there is no theoretical limitation on the height of dam. As for central core earth and rockfill construction, space is restricted for dams less than about 15 to 20 m high, which makes construction complicated and other design types may be more economic.

h) Concrete face rockfill. Constructed ideally of free draining rockfill, with a 'cushion' layer of finer processed rockfill between the concrete faceplate and the rockfill to provide uniform support for the faceplate. The face is formed on the rockfill by slip forming (usually after rockfill placement is complete), is reinforced and provided with construction joints and water stops to control cracking under the deformations induced by the water load. Poorer quality rockfill has been used, with zones of free draining rock incorporated to allow discharge of leakage. Should this occur, as the rockfill is free draining, pore pressures do not develop in the dam, and the upstream and downstream slopes are commonly at the angle of repose of the rockfill. The cushion layer is commonly designed to limit leakage if the faceplate cracks or construction joints open and, with zoning of the rockfill, to act as a filter relative to the rockfill concrete face. Rockfill (CFRF) dams up to 160 m high are in service and larger dams are under construction. For dams less than about 20 m high the cost of the plinth and setting up for faceplate construction usually makes concrete face rockfill uneconomic. CFRF dams are particularly suited to construction in high rainfall areas, where placement of earthfill is impracticable, and to sites with good rock foundations.

i) Bituminous concrete face earth and rockfill. Constructed ideally of free draining rock or gravel/sand fill, but a wide range of earth and rockfill has been used the bituminous concrete facing is constructed *in situ* on the fill, and usually consists of a 'sandwich structure' with at least two layers of dense bituminous concrete to act as the 'impervious' membrane, and a bituminous concrete drainage layer (ICOLD 1982a). Seepage and piping control is provided by the bituminous concrete, and drainage layers in the fill. Bituminous concrete face dams up to 50 m high have been constructed but this design is more commonly applied to dams and reservoirs of lesser height. Bituminous concrete has been used as a core material in a limited number of earth and rockfill dams, usually where there is little earthfill available. Details are given in ICOLD 1982b).

j) Steel face rockfill. Constructed as for concrete face rockfill, but with a steel upstream face. The plate is provided with expansion joints and cathodic protection to control corrosion. Only limited use has been made of this type of dam construction because of economics and concern about corrosion. Dams up to 40 m high have been built.

k) Thin membrane face earth and rockfill. Constructed ideally of sand, sand and gravel or rockfill, with a thin membrane placed on the upstream face membranes of elastomers (isobutylene), plastomers (low density polyethylene, polyvinyl chloride), of thickness 0.2 mm to 2 m and bituminous from 3 to 8 mm have been used (ICOLD, 1981). The membrane is supported on a sand or sand and gravel cushion and covered by filter fabric, sand, rockfill or fabricated layer slabs to protect the membrane from damage by sunlight, wave action and floating debris. ICOLD (1981) recommends that thin membranes be restricted to dams less than 30 m high. It seems likely that this design will be used for higher dams as experience is gained with the use of membranes.

1.2 ZONING OF EMBANKMENT DAMS AND TYPICAL CONSTRUCTION MATERIALS

1.2.1 *General principles*

The following discussion describes 'typical' zoning and construction materials for the most common types of embankment dams. Each dam should be designed to satisfy the particular topographic and foundation conditions at the site and to use available construction materials, so there really are no 'typical' or 'standard' designs. To highlight this, examples are given of the various types of dams. Many of these are drawn from Australian practice as detailed in the ANCOLD (Australian National Committee on Large Dams) Bulletins, and from the authors' own experience. Figure 1.3 shows typical cross sections for the most common types of zoned embankment dams. Table 1.2 describes the zoning numbering system used in Figure 1.3 (and throughout this book) and the function of the zones. Table 1.3 describes terms used relating to foundation treatment.

Table 1.2. Embankment dam zone description and function.

Zone	Description	Function
1	Earthfill	Controls seepage through the dam
2A	Fine filter (or filter drain)	(a) Prevent erosion of Zone 1 by seepage water, (b) Prevent erosion of the dam foundation (where used as horizontal drain), (c) Prevent buildup of pore pressure in downstream face when used as vertical drain
2B	Coarse filter (or filter drain)	(a) Prevent erosion of Zone 2A into rockfill, (b) Discharge seepage water collected in vertical or horizontal drain
2C	(i) Upstream filter	Prevent erosion of Zone 1 into rockfill upstream of dam core
	(ii) Filter under Rip Rap	Prevent erosion of Zone 1 through rip rap
2D	Fine cushion layer	Provide uniform support for concrete faceplate; limit leakage in the event of the faceplate cracking or joints opening
2E	Coarse cushion layer	Provide uniform support for concrete faceplate. Prevent erosion of Zone 2D into rockfill in the event of leakage
3A	Rockfill	Provides stability, commonly free draining to allow discharge of seepage through and under the dam. Prevents erosion of Zone 2B into coarse rockfill
3B	Coarse rockfill	Provides stability, commonly free draining to allow discharge of seepage through and under the dam
4	Rip rap	Prevents erosion of the upstream face by wave action

Table 1.3. Embankment dam foundation treatment.

Item	Description
General excavation	Excavation of compressible and low strength soil and weathered rock as is necessary to form a surface sufficiently strong to support the dam and to limit settlement to acceptable values
Cuttoff excavation	Excavation below general excavation level to remove highly permeable soil and rock

Table 1.4 describes typical construction materials used for the different zones in embankment dams. It is emphasised that good dam engineering involves use of the materials available at the site rather than to look for materials with a preconceived ideas about the material properties needed. However this rule is not followed in the search for Zone 2A and 2B filters where invariably one might seek close particle size grading limits and dense hard durable materials.

Penman (1982, 1983) reinforces this point and describes materials commonly used for embankment construction.

1.2.2 *Examples of embankment designs*

1.2.2.1 *Zoned earthfill dams*

Figures 1.4, 1.5 and 1.6 show examples of zoned earthfill design. All were designed for low hazard sites. The Tahmoor Dam site was underlain by sandstone, and there were limited sources of earthfill, which was not dispersive. The sandstone rock breaks down to a silty sand when

Table 1.4. Embankment dam typical construction materials.

Zone	Description	Construction materials
1	Earthfill	Clay, sandy clay, clayey sand, silty sand, possibly with some gravel. Usually more than 15% passing 75 µm, preferably more. Note that weathered siltstone, shale and sandstone can be compacted in thin layers to give sufficiently fine material
2A	Fine filter	Sand or gravelly sand, with less than 5% (preferably less than 2%) fines passing 75 µm. Fines should be non plastic. Manufactured by crushing, washing, screening and recombining sand-gravel deposits and/or quaried rock
2B	Coarse filter	Gravelly sand or sandy gravel, manufactured as for Zone 2A. Zones 2A and 2B are required to be dense, hard durable aggregates with similar requirements to that specified for concrete aggregates. They are designed to strict particle size grading limits to act as filters
2C	Upstream filter and filter under rip rap	Sandy gravel/gravelly sand, well graded, 100% passing 75 mm, not greater than 8% passing 75 µm, fines non plastic. Usually obtained as crusher run or gravel pit run with a minimum of washing, screening and regrading. Relaxed durability and filter design requirements compared to Zones 2A and 2B
2D	Fine cushion layer	Silty sandy gravel well graded, preferably with 2-12% passing 75 µm to reduce permeability (Sherard 1985). Obtained by crushing and screening rock or naturally occurring gravels or as crusher run. Larger particles up to 200 mm are allowed by some authorities (Fitzpatrick et al. 1985)
2E	Coarse cushion layer	Fine rockfill placed in 500 mm layers to result in a well graded sand/gravel/cobbles mix which satisfies filter grading requirements compared to Zone 2D
3A	Rockfill	Quarry run rockfill, possibly with oversize removed in quarry or on dam. Preferably dense, strong, free draining after compaction, but lesser properties are often accepted. Compacted in 0.5 to 1 metre layers with maximum particle size equal to compacted layer thickness
3B	Coarse rockfill	Quarry run rockfill. Preferably dense, strong, free draining after compaction, but lesser properties are often accepted. Compacted in 1.0 to 1.5 m layers with maximum particle size equal to compacted layer thickness
4	Rip rap	Selected dense durable rockfill sized to prevent erosion by wave action. In earth and rockfill dams often constructed by sorting larger rocks from adjacent 3A and 3B zones. In earthfill dams either selected rockfill or a wider zone of quarry run rockfill

compacted in thin layers, yielding an acceptable transition filter material.

The Blair Athol Dam was built over alluvium and an existing dam, which had been built without conventional design or construction control. A cutoff trench was excavated to the weathered sandstone and siltstone foundation. Zoned earthfill construction was practicable, using a transition zone of weathered basalt rock compacted in 150 mm thick layers. The 'earthfill' was sandstone from mine overburden compacted in 150 mm thick layers.

The silt trap dam at the Argyle Mine is founded on a moderately weathered basalt founda-

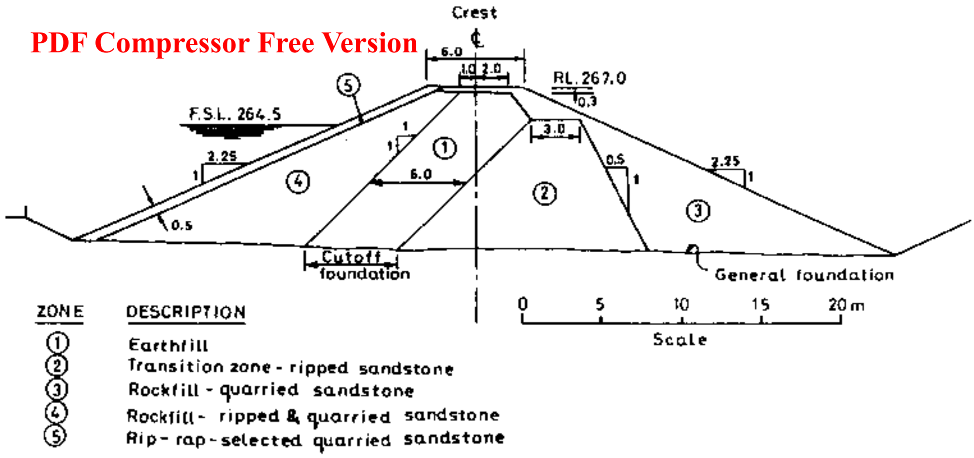


Figure 1.4. Mine water supply Tahmoor Dam (courtesy of Coffey Partners International).

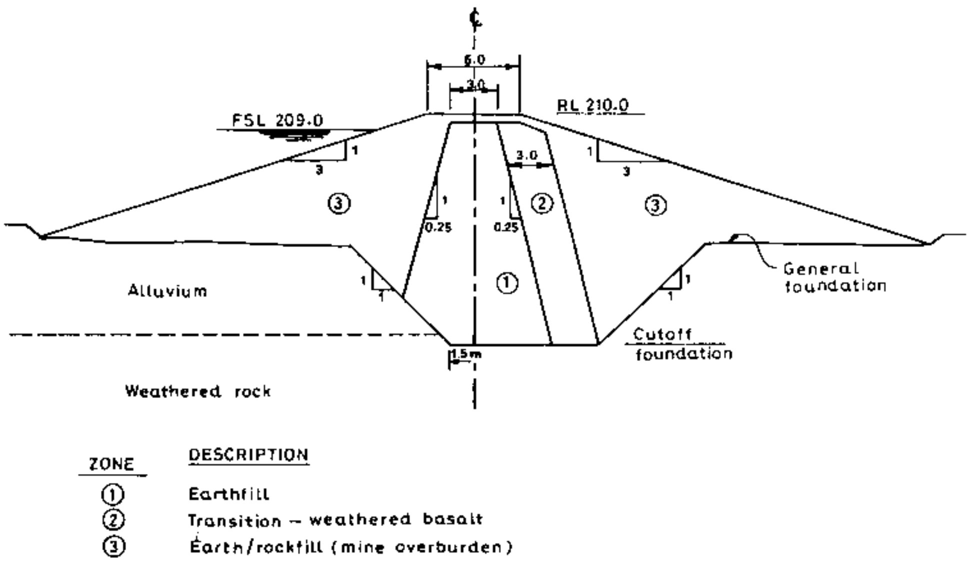


Figure 1.5. Water storage dam, Blair Athol Mine (courtesy of Coffey Partners International).

tion. It is an unusual dam, in that it is meant to allow water to flow through it, with silt and sand runoff from the mining operations being collected in the storage.

1.2.2.2 *Earthfill dams with horizontal and vertical drains*

The mine runoff water dam at Drayton Mine is constructed on a foundation of interbedded siltstone and sandstone. It is a low hazard site, allowing adoption of a design without a vertical drain. Zone 3 was compacted mine overburden (mainly sandstone), which results in a low

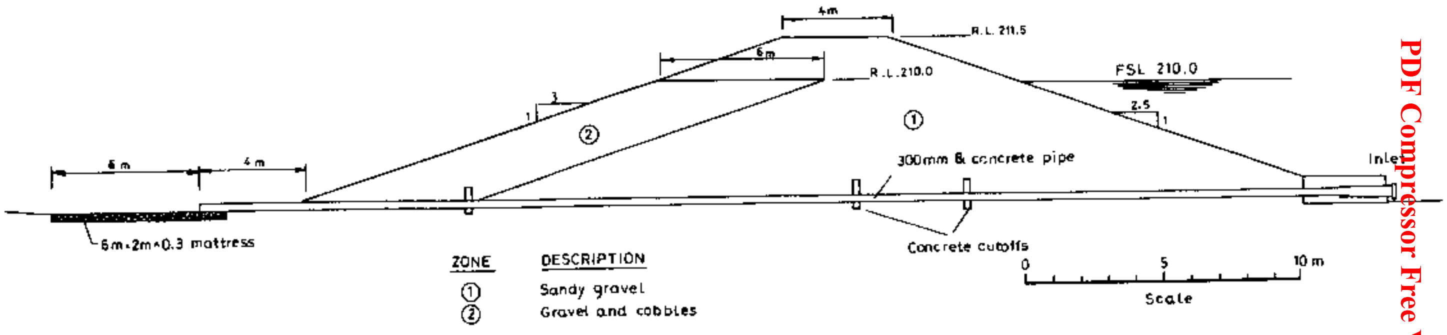


Figure 1.6. Silt trap dam, Argyle Mine (courtesy of Coffey and Partners).

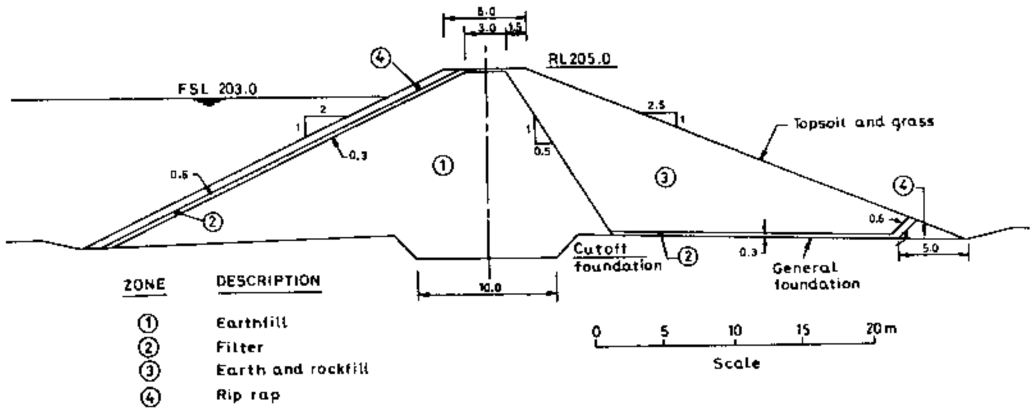


Figure 1.7. Mine runoff water dam, Drayton Mine (courtesy of Coffey and Partners).

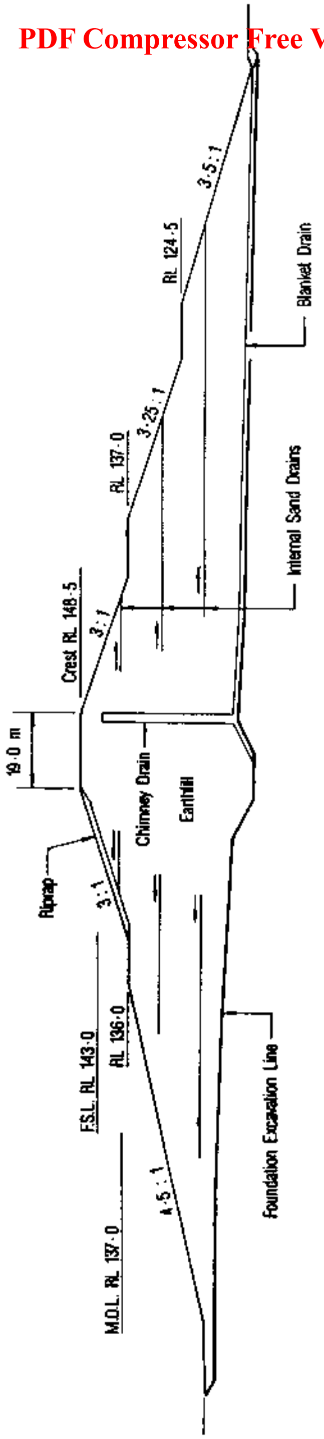


Figure 1.8. Ash Pond Dam, Loy Yang Power Station (courtesy State Electricity Commission of Victoria).

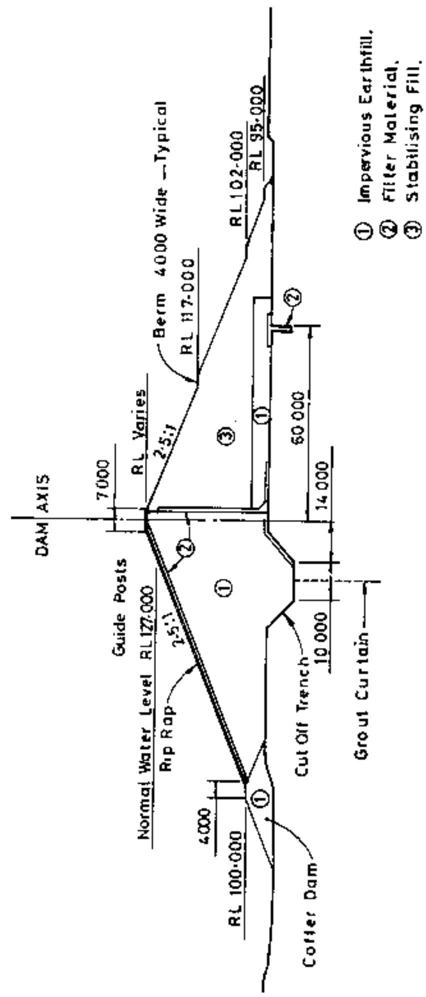
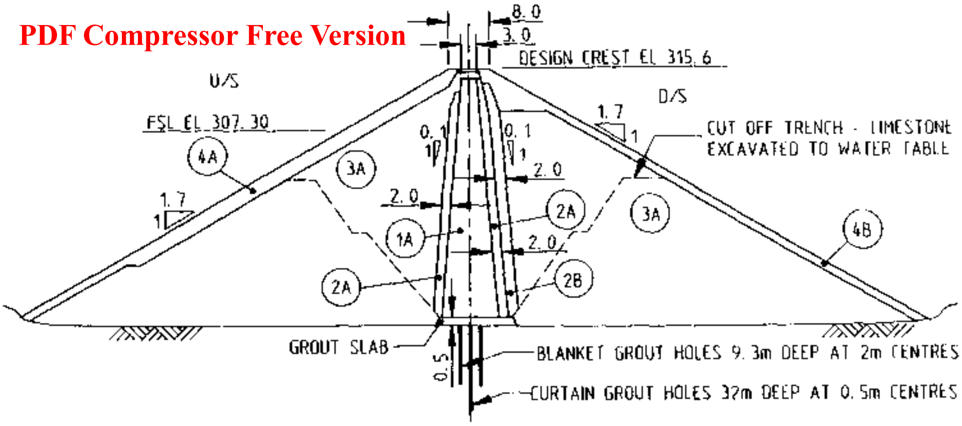


Figure 1.9. Plashett Dam (ANCOLD, 1990).



ZONE	DESCRIPTION
1A	CLAY CORE
2A	FINE FILTER
2B	COARSE FILTER

ZONE	DESCRIPTION
3A	ROLLED ROCKFILL
4A	UPSTREAM RIP RAP
4B	DOWNSTREAM RIP RAP

Figure 1.10. Bjelke-Petersen Dam (courtesy of Water Resources Commission of Queensland).

permeability earth/rockfill. A horizontal drain was provided to collect seepage through the foundation in a controlled manner.

The Ash Pond Dam for Loy Yang Power Station is founded on Quarternary sediments overlying Tertiary coal measures.

The sediments include widely interspersed beds of overconsolidated silty and sandy clays, and clayey and silty sands. The coal measures consist of inferior coal and coal seams separated by interseams, composed of sandy clays, silts and silty sands which form a series of aquifers. The dam has a vertical chimney drain and horizontal drain, to control seepage through the dam and its foundation. Internal sand drains were provided to control construction pore pressures. Increased piezometric levels within these aquifers following filling of the ash storage, required the installation of a series of relief wells at the downstream toe of the dam.

Plashett Dam is founded on coal measure rocks including sandstone, siltstone and coal. The design incorporates a cutoff through alluvial soils in the foundation, a vertical drain, and horizontal drain provided as strips. A vertical drain is provided through the sediments to intercept seepage which bypasses the cutoff.

1.2.2.3 *Central core earth and rockfill dams*

The Bjelke-Petersen dam is largely founded on a very complex sequence of andesitic volcanic rock and limestones, which have been strongly sheared and folded. These sequences alternate with metamorphosed phyllites, steeply dipping fault zones occurring at the contacts between sequences. The limestone exhibited solution channels and sink holes, often infilled with stiff high plasticity clay. These were less frequent and less permeable below the water table.

The cutoff trench was excavated through the limestone to the water table, and a reinforced concrete grout slab formed. A high pressure grout curtain, with close hole spacing, was adopted.

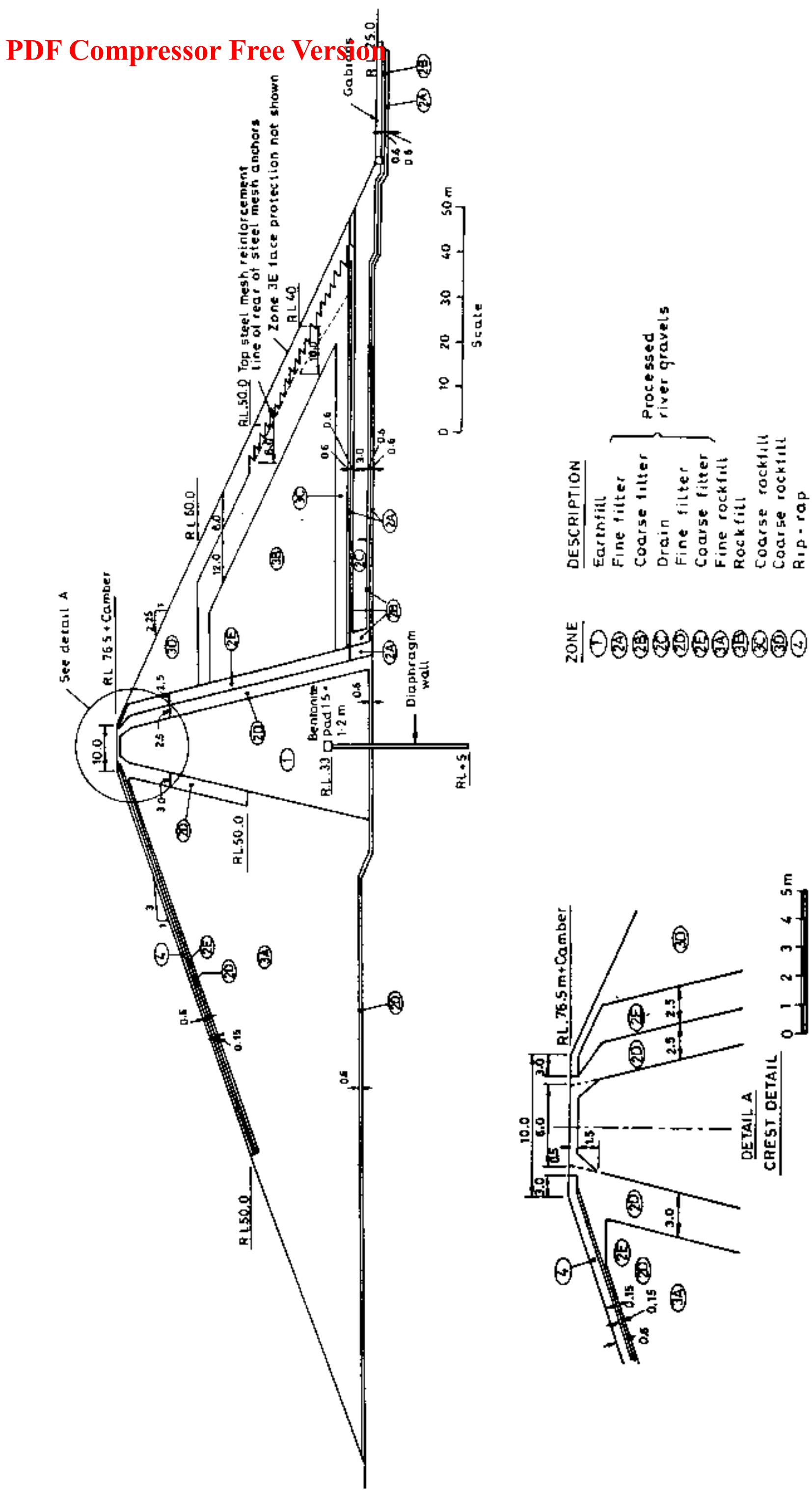


Figure 1.11. Proposed Lungga Dam (courtesy of Coffey Partners International).

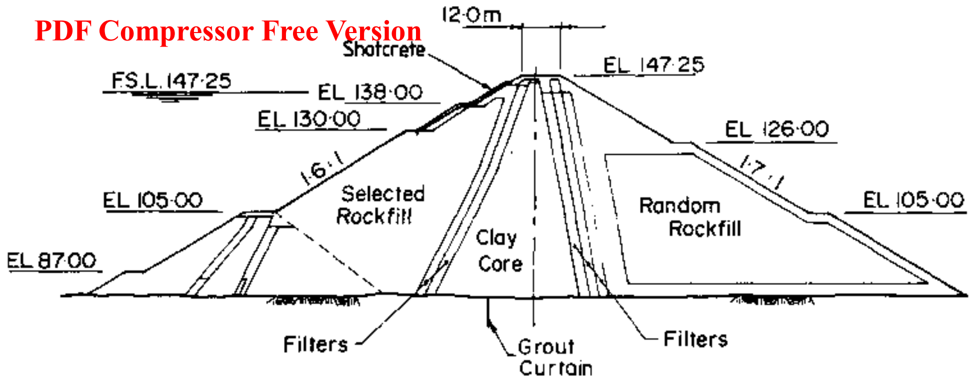


Figure 1.12. Blue Rock Dam (ANCOLD, 1990).

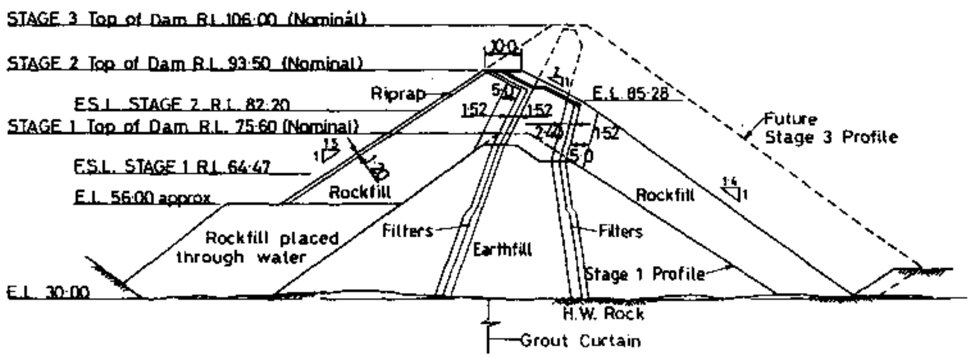


Figure 1.13. Hinze Dam (ANCOLD, 1990).

The core was kept narrow to minimise the cost of excavation and foundation treatment McMahon (1986) gives more details of the project.

The Lungga Dam, in the Solomon Islands, was to have been constructed on 60 m of alluvial sand and gravel. A partially penetrating diaphragm wall was to assist in reducing seepage flow in the foundation, but substantial seepage was still anticipated. The available rockfill was relatively low permeability, so a substantial horizontal drain and protective filters was incorporated into the design.

Blue Rock Dam is a fairly conventional central core earth and rockfill dam, constructed on a foundation of Silurian mudstones with thin interbedded sandstones. An unusual feature of the embankment is the use of reinforced shotcrete on the upstream face for wave protection, as the available rock was of inadequate size and quality. This is discussed further in Chapter 13 (Fig. 13.9).

Hinze Dam is a central core earth and rockfill dam which is founded on greywacke, greenstone and chert. It is unusual (for a central core dam) in that it has been designed to be constructed in three stages. This leads to a modified form of geometry, and necessitates placing some rockfill under water.

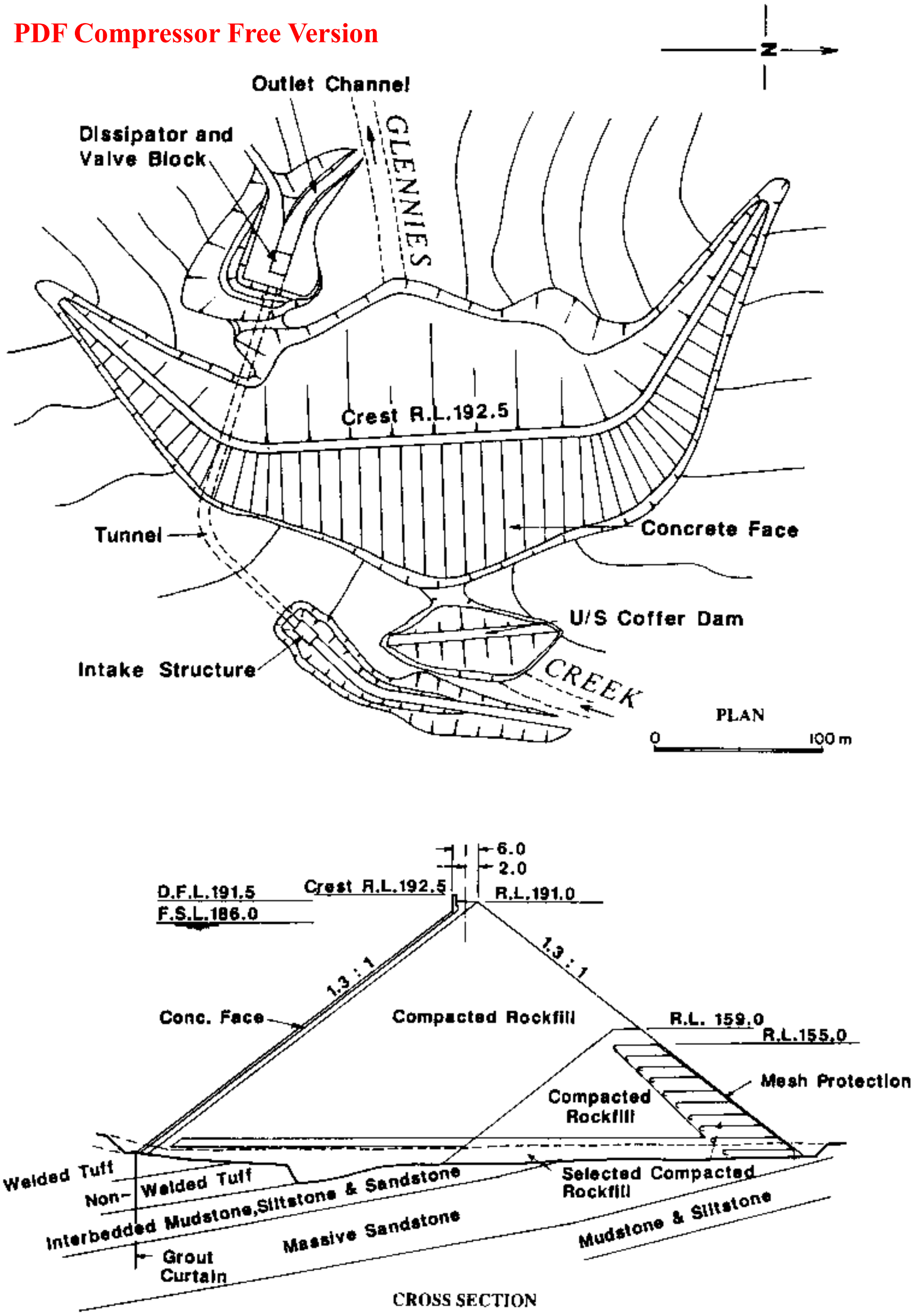


Figure 1.14. Glennies Creek Dam (ANCOLD, 1990).

1.2.2.4 *Sloping upstream core earth and rockfill dam*

Examples of sloping upstream core dams are given in Figures 17.30 and 17.31. In both cases, the layout has been adapted to facilitate staged construction.

1.2.2.5 *Concrete face rockfill dams*

Several examples of concrete face rockfill dams are given in Chapter 16, including Figures 16.1, 16.2, 16.3, 16.4, 16.5 and 16.22. Figure 1.14 shows the design of Glennies Creek Dam.

The dam was founded on welded and non welded ash flow tuff overlying interbedded sandstones, siltstones and mudstones. The axis of the dam was curved, to allow the plinth for the concrete face to be founded on the welded tuff.

At the site of Reece Dam, the right abutment is strong laminated quartzite. The river bed is infilled with fluvioglacial gravels, which were left in place under the rockfill. The left abutment is schist and amphibolite, weathered to a depth of 30 metres, and special detailing was necessary to found the plinth on this weathered rock. This is discussed in more detail in Chapter 16 (Fig. 16.24).

1.3 SELECTION OF EMBANKMENT TYPE

The selection of the type of dam embankment to be used at a particular site is affected by many factors, some of which are outlined below. The dam engineer's task is to consider these factors and adopt a suitable design. The overriding consideration in most cases will be to construct an adequately safe structure for the lowest total cost. Hence, preparation of alternative designs and estimates of cost for those alternatives will be a normal part of the design procedure. Usually the most economic design will be that which uses a construction materials source close to the dam, without excessive modification from the 'borrow pit run' or 'quarry run' material.

The following outlines some of the factors involved and their effect on embankment type.

1.3.1 *Availability of construction materials*

1.3.1.1 *Earthfill*

Clearly the availability of suitable earthfill within economic haul distance is critical in the selection of the embankment type. If there is no earthfill available – for example in an area underlain by sandstone, which weathers to give only a shallow cover of sandy soil, it will normally be appropriate to construct a rockfill dam with concrete (or other) impervious membrane.

The uniformity of the available earthfill will also influence design, and the method of construction. If the borrow areas produce two different types of earthfill, the earthfill may be zoned into two parts, e.g. for an earthfill dam with vertical and horizontal drains, the earthfill with lower proportion of fines and more variable properties would be best placed downstream of the vertical drain or the earthfill zone may be separated into Zones 1A and 1B with the coarser soil in Zone 1B adjacent the Zone 2A filter.

Alluvial clayey soils are often more variable than residual soils derived from weathering of underlying rocks and, hence, it may be necessary to provide additional zoning as described above. Alternatively, the soils may be mixed by borrowing from a vertical face with a shovel and truck operation (rather than use of scrapers). Blending of soils on the dam embankment is generally avoided as it leads to increased cost.

If cobbles and boulders are present in clayey soil deposits, these will have to be removed prior to compaction either by passing the earthfill through a 'grizzly' or by grader or hand labour on the embankment. This is necessary to prevent the oversize particles affecting compaction.

Relatively permeable soils can be used for earthfill in many dam projects. The permeability of most dam foundations is between 1 and 10 Lugeons (10^{-7} to 10^{-6} m/sec) so for most dams an earthfill core with permeability, say 10^{-9} m/sec, seepage through the foundations will far exceed that through the dam. Even if silty sand is used for the earthfill, a permeability of 10^{-6} m/sec should be achieved, i.e. not higher than the foundation permeability, and from a seepage viewpoint such a high permeability soil would be acceptable. Weathered or even relatively unweathered siltstone and sandstone with a clayey matrix, may break down sufficiently when compacted in thin layers to achieve a satisfactory core material. This will usually require field trails to observe the actual properties. MacKenzie & McDonald (1985) and ANCOLD (1985) describe trials and actual performance of compacted siltstone and sandstone, during construction of Mangrove Creek concrete face rockfill dam.

1.3.1.2 *Rockfill*

Over the last 20 years the unit rate for construction of rockfill has reduced compared to that for earthfill, and as usually less rockfill is required (because the side slopes can be steeper), the availability of rock which can be quarried to yield free draining rockfill often leads to economic dam design.

Most igneous and many metamorphic rocks, e.g. granodorite, diorite, granite, basalt, rhyolite, andesite, marble, greywacke, quartzite will when fresh, yield free draining rockfill.

Some metamorphic rocks, e.g. phyllite, schist, gneiss and slate, may break down under compaction to yield a poorly draining rockfill even though it may be dense with a high modulus. The amount of breakdown depends upon the degree to which foliation or cleavage is developed in the rocks.

Most highly weathered, and many moderately weathered igneous and metamorphic rocks, will not yield totally free draining rockfill.

The spacing of the bedding and joint planes influences the size and grading of rockfill obtained from a quarry. Blasting may be varied to yield the required sized product, but this is not always practicable.

Thick beds of say sandstone within a sequence of thinner bedded siltstone and sandstone, are likely to yield oversize rock which would require either secondary breaking in the quarry, or sorting and disposal on the embankment.

Often a substantial amount of the rockfill for a dam will come from 'required excavations,' i.e. from the spillway, foundation for the dam, inlet and outlet works etc. This is in principle desirable as the effective cost of the rockfill could be only the cost of placement (and any additional haulage costs). However, the rock quality from these excavations may not be ideal (due to rock type, weathering, method of excavation) necessitating changes to the embankment zoning to accommodate the material. The timing of production of rock from required excavations may not be ideal from the viewpoint of dam construction and in the event, may not be as scheduled. Some flexibility in zoning is desirable to allow for such circumstances. Such flexibility is also useful to allow for different quality of rock being obtained from required excavations, than that anticipated at the time of designing the embankment.

Most sedimentary rocks, e.g. sandstone, siltstone, shale, mudstone generally tend to break down under compaction even when fresh and yield poorly draining rockfill. In these circumstances it may be necessary to incorporate zones of free draining rockfill to ensure the embankment rockfill as a whole is capable of remaining free draining, e.g. Figures 1.11, 1.14 and 16.22.

The use of so-called random rockfill zones in a dam section also facilitates use of rock, which does not yield free draining rockfill.

1.3.1.3 *Filters*

A source of high quality sand and gravel is necessary for construction of filters. It may be necessary to obtain these materials from many kilometres distance (50 km is not unusual), despite the haulage costs.

Filter aggregates may be obtained from alluvial sand and gravel deposits, or from quarries. Generally suitable aggregates are of igneous and less commonly metamorphic origin. It is very unusual to manufacture from sedimentary rocks, as these rocks are usually not sufficiently durable and often have poor shape (as measured by flakiness index).

For large dams it is usually necessary to establish a separate crushing and screening plant for manufacture of filter and concrete aggregates. It is sometimes necessary to let a separate early contract to begin the manufacture and stockpiling of aggregates as their production rate controls dam construction progress.

Where sources of filter aggregates are far from the dam and for this or other reasons filters are expensive, the width of filter zones may be reduced by using spreader boxes etc. (see Chapter 7). Such a situation may also favour use of concrete face rockfill, rather than earth and rockfill construction.

1.3.2 *Foundation conditions*

The strength, permeability and compressibility of the dam foundation have a major influence on the embankment type, e.g:

– a soil foundation will have a relatively low strength, which may determine the embankment stability, and will require relatively flat embankment slopes. Such a foundation is likely to favour construction of earthfill dams, i.e. earthfill with horizontal and vertical drains, rather than rockfill;

– a permeable soil foundation will be susceptible to leakage and erosion, requiring construction of some form of cutoff, and a filter drain under the downstream slope of the dam (see Chapter 9);

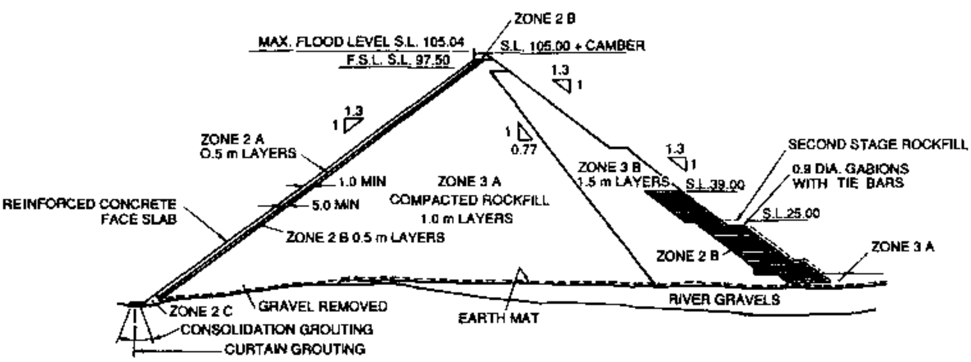
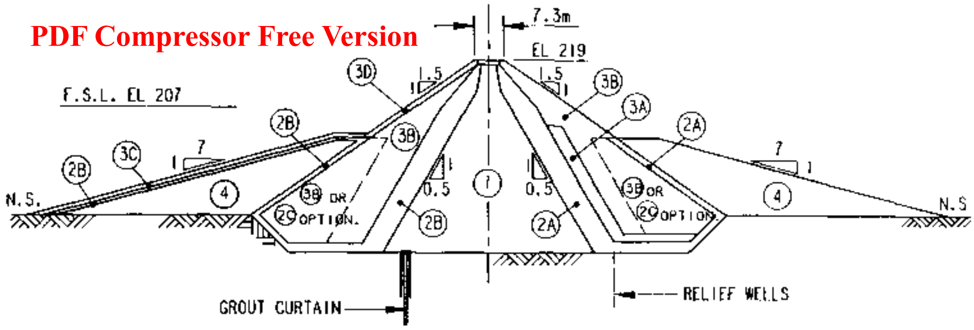


Figure 1.15. Reece Dam (also known as Lower Pieman Dam) (ANCOLD 1990).



ZONE	DESCRIPTION
1	SELECTED CORE MATERIAL
2A	WASHED GRAVELLY SAND
2B	PIT RUN GRAVELLY SAND
2C	FREE DRAINING GRAVELLY SAND

ZONE	DESCRIPTION
3A	ROCKFILL - QUARRY FINES
3B	ROCKFILL
3C	RIP RAP
3D	RIP RAP (SEE SPEC.)
4	MISCELLANEOUS FILL

Figure 1.16. Maroon Dam (courtesy of Water Resources Commission of Queensland. Note: Drawing is not to scale).

– a strong low permeability rock foundation is suited to any type of dam construction, but may favour construction of a concrete face, concrete gravity, or in particular cases concrete arch dam;

– in earthquake zone areas, the presence of loose to medium dense saturated sandy soils in the foundation will be important, as liquefaction may occur during earthquakes. This may necessitate densification of sandy soil and/or the provision of weighting berms;

– dams on karst limestone foundations are a special case, where extensive grouting and other work may be needed to limit leakage to acceptable levels. Such situations favour adoption of a design, which allows for grouting to continue during embankment construction or after it is completed. This may tend to favour concrete face rockfill or earth and rockfill with a sloping upstream core;

– in some sedimentary rocks, particularly interbedded weak claystone and mudstone, and strong sandstone, which have been subject to folding and/or faulting, bedding plane shears may exist, resulting in low effective friction angles (Hutchinson 1988, Casinder 1980)). In these cases, flat slopes may be required on the embankment, favouring earthfill with vertical and horizontal drains, or earth and rockfill with random rockfill zones. Figure 1.16 shows the cross section for Maroon Dam, where such conditions required very flat slopes;

– in some areas, often, but not always tropical, the rock is deeply weathered and sometimes with a lateritic profile which may lead to a high permeability, soil strength foundation, favouring embankments with flatter slopes and good under drainage, eg. earthfill with vertical and horizontal drain;

– embankments constructed on deep soils, e.g. deep alluvium in or adjacent to the river bed, may be subject to a large amount of settlement, leading to differential movement and cracking. In such dams it is particularly important to provide good filters to control seepage and prevent internal erosion.

1.3.3 *Climate*

It is difficult (and in many cases impossible) to construct earthfill embankments during wet weather, or in freezing temperatures. This is particularly critical when the rain is relatively continuous without high evaporation (and not so critical when the rain is in short storms, followed by hot sunshine).

In these circumstances, it is often advantageous to adopt concrete face rockfill or sloping upstream core construction, so that the rockfill can continue to be placed in the wet weather, and the faceplate or core constructed when the weather is favourable.

In very arid areas there may be a shortage of water for construction, thus favouring concrete face rockfill rather than earthfill.

1.3.4 *Topography and relation to otherstructures*

The selection of embankment type and overall economics of a project is determined with consideration of all components of the project, i.e. embankment, spillway, river diversion outlet works etc. These components are interrelated, e.g.:

– the diversion tunnel will be longer for an earthfill dam (with relatively flat side slopes), than for a rockfill dam;

– the spillway will generate rockfill (and possibly earthfill and random fill) for use in the embankments. The size of spillway can be varied by storing more floodwater in the reservoir, necessitating a higher embankment but smaller spillway, so the optimisation of total project cost may influence embankment zoning and size;

– it is common practice in Australia to allow large floods during construction to pass over the embankment, rather than providing a larger diversion tunnel and coffer dam. This necessitates incorporation of some rockfill in the downstream toe of the dam with steel mesh reinforcement (see Chapter 13).

The topography of the site, i.e. valley cross section slope curve of the river in plan, presence of 'saddles' in the abutments, can have a significant effect on embankment selection, e.g.:

– in narrow steep sided valleys there is restricted room for construction vehicles and haul roads, favouring embankments with simple zoning, e.g. concrete face rockfill;

– the curve of the river in plan, and changes in valley cross section, may favour adoption of an upstream sloping core rather than central core (or vice versa) to reduce the quantities of earthfill;

– local changes in slope of the abutments may lead to differential settlement and cracking, necessitating more extensive filter drains, or favouring concrete face rockfill construction.

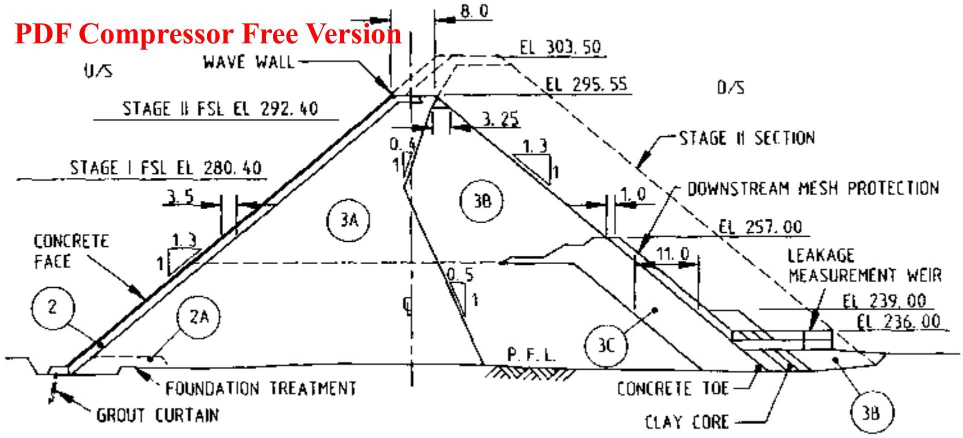
1.3.5 *Staged construction*

It is often economic to construct a dam in two or more stages, e.g.:

– in water supply, irrigation or hydropower projects, demand in the early years can often be met with a lower dam and smaller storage;

– in mine tailings dams, the storage required increases progressively as the tailings are deposited in the dam.

If staging is planned, this favours adoption of concrete face rockfill, earth and rockfill with sloping upstream core, or possibly earth fill with vertical (or sloping) drain and horizontal drain. Figures 1.13, 1.17, 16.22, 17.30 and 17.31, show examples of dams designed for staged construction.



ZONE	DESCRIPTION
2	FACE MATERIAL
2A	FILTER MATERIAL

ZONE	DESCRIPTION
3A	ROCKFILL-UP TO 600mm DIA.
3B	ROCKFILL-UP TO 900mm DIA.
3C	AS PER 3B WITH NO MORE THAN 5% PASSING THROUGH A 100mm APERTURE

Figure 1.1.7. Boondooma Dam (courtesy Water Resources Commission of Queensland).

1.3.6 *Time for construction*

The time available for construction may influence the selection of dam type, particularly if considered in relation to other factors such as the climate, e.g. in a climate of well defined wet and dry seasons it may be practicable to construct an earth and rockfill or earthfill dam, but only over two dry seasons. A concrete face rockfill dam may be constructed in lesser time by continuing to place rockfill in the wet season.

Foundation treatment and zoning details may also be influenced by the time available for construction. For example, if constructing a dam on a permeable soil foundation, cutoff may be achieved by a cutoff wall at the upstream toe rather than a rolled earth cutoff under the central core, so that the cutoff wall can be constructed at the same time as the rest of the embankment.

CHAPTER 2

Weathering processes and profiles in valleys

Most dams and storages are located at or near the bases of river valleys which have been formed by the complex interaction of various processes of weathering (breakdown) and erosion (removal) of rocks and soils. For a general understanding of weathering and erosional processes the reader is referred to Selby (1982), Hunt (1984) and Bell (1983). Unfortunately none of these texts gives adequate description or explanation of some important features commonly encountered in dam foundations. Such features will be described here.

2.1 HIGH HORIZONTAL STRESSES IN ROCK

Measurements of insitu rock stresses at shallow depths in many geological environments throughout the world have shown horizontal stresses which are generally higher than can be explained theoretically from the weight of the present overburden. Hast (1967) published the results of a large number of stress measurements in relatively intact granitic and metamorphic rocks in Scandinavia. Hast showed that at all Scandinavian sites, the major and intermediate principal stresses were horizontal, and that the sum of these two stresses (Line A on Fig. 2.1) increased linearly with depth. Also plotted on Figure 2.1 are the mean horizontal stress (Line B) and the theoretical vertical stress due to overburden weight (Line C). It can be seen that within about 50 m of the ground surface the mean horizontal stress appears likely to be more than ten times the vertical stress.

Brown & Hoek (1978) compiled Figures 2.2 and 2.3 from numerous stress measurements at mining and civil engineering sites throughout the world. Figure 2.2 shows that the vertical components of the measured stresses are in fair agreement with the calculated vertical stress due to overburden. Figure 2.3 shows the variation with depth, of the ratio of the mean horizontal stress to the vertical stress. It can be seen that horizontal stresses are generally significantly greater than vertical stresses, at depths of less than 500 m, and that at greater depths the stresses tend to equalize.

2.1.1 *Probable cause of high horizontal stresses*

The high measured stresses in most situations are believed to result from strain energy which has been locked into rocks during their formation, often, but not necessarily, at great depth. This has occurred in igneous rocks during their solidification, and in sedimentary and metamorphic rocks, during compaction, cementation or recrystallization; Emery (1964), Savage (1978),

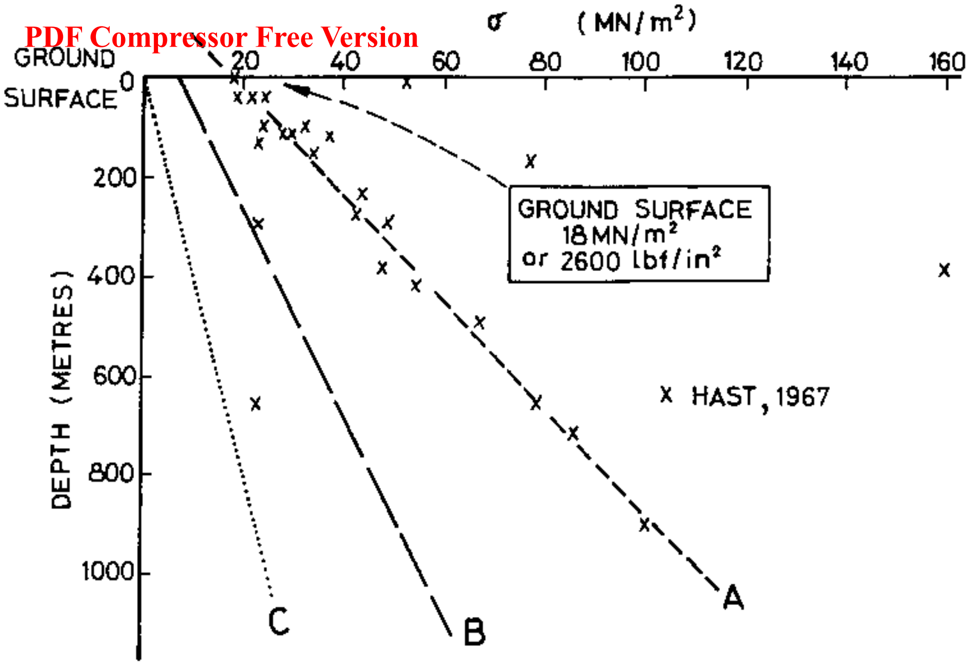


Figure 2.1. Plot of horizontal stresses against depth in Scandinavian mines (based on Hast 1967).

Nicholls (1980) and Brady & Brown (1985). At a microscopic level the strain energy is considered to be locked into mineral crystals or grains, by cementation and interlocking. An analogous model would be compressed springs embedded in plastic (Fig. 2.4).

As the vertical load on highly stressed rock is slowly lowered by erosion (Fig. 2.5) vertical stresses are relieved progressively by upward expansion. However, because the rock remains confined laterally, the horizontal stresses decrease in accord with Poissons Ratio, i.e., at about one-third of the rate of the vertical stresses. This results in the near-surface imbalance shown on Figures 2.1 and 2.3.

2.1.2 *Stress relief effects in natural rock exposures*

Field evidence of the existence of the high horizontal stresses at shallow depths is seen most clearly in areas of massive igneous rocks which contain very few tectonically induced fractures. Most of Hast's measurements plotted on Figure 2.1 were made in such rocks. These rocks usually contain 'sheet joints' near-parallel to the ground surface, as shown in Figure 2.6. The sheet joints show rough, irregular, plumose surfaces indicating that the rock failed in tension, by buckling or spalling, the tension being induced by the high horizontal compressive stress. Some sheet joints show slickensides near their extremities indicating local shear failure which should be expected here (Fig. 2.6). The spacing of sheet joints is often 0.3 to 1 m near the ground surface, becoming progressively wider with depth.

From all of these characteristics, sheet joints are considered to be stress-relief features. The linear decrease in horizontal stresses as the ground surface is approached (Lines A and B, Fig.

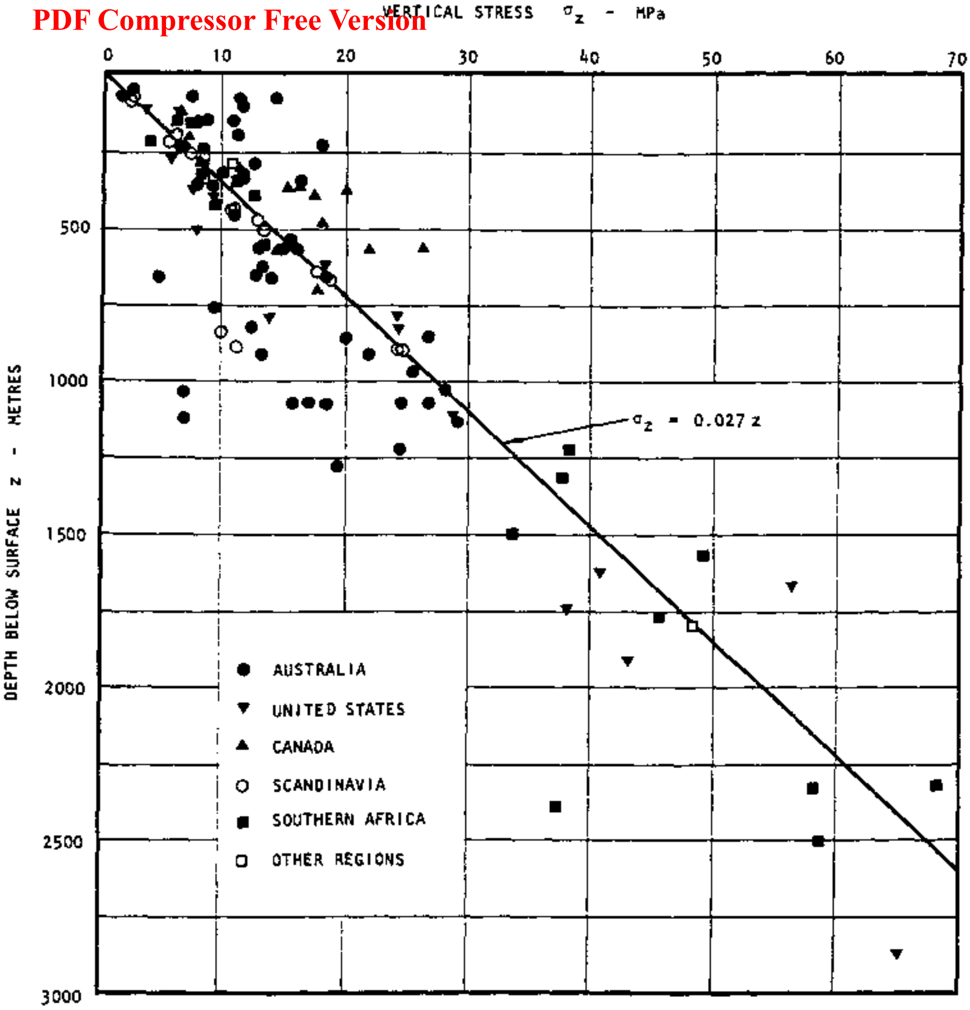


Figure 2.2. Plot of vertical stresses against depth below ground surface (from Brown & Hoek 1978, by permission of Pergamon Press).

2.1) is due to the progressive relief of horizontal stresses partly by buckling and spalling, as the overlying rock load is removed by erosion.

Holzhausen (1989) provides a comprehensive account of the characteristics and origin of sheet joints.

In fractured rock masses, i.e. those which are already weakened by defects of tectonic origin, the effects of horizontal stress relief are not quite so obvious. However, the destressing effects are always present, as opening up of the existing defects, as shown in Figure 2.7b. The destressing effects usually extend to greater depths in fractured rock than in massive rock.

Claystones and shales (i.e. 'mudrocks') are formed mainly by consolidation of clay-rich sediments, but may be strengthened further by partial recrystallization and cementation. Such rocks have much higher porosities than igneous and metamorphic rocks. They usually show

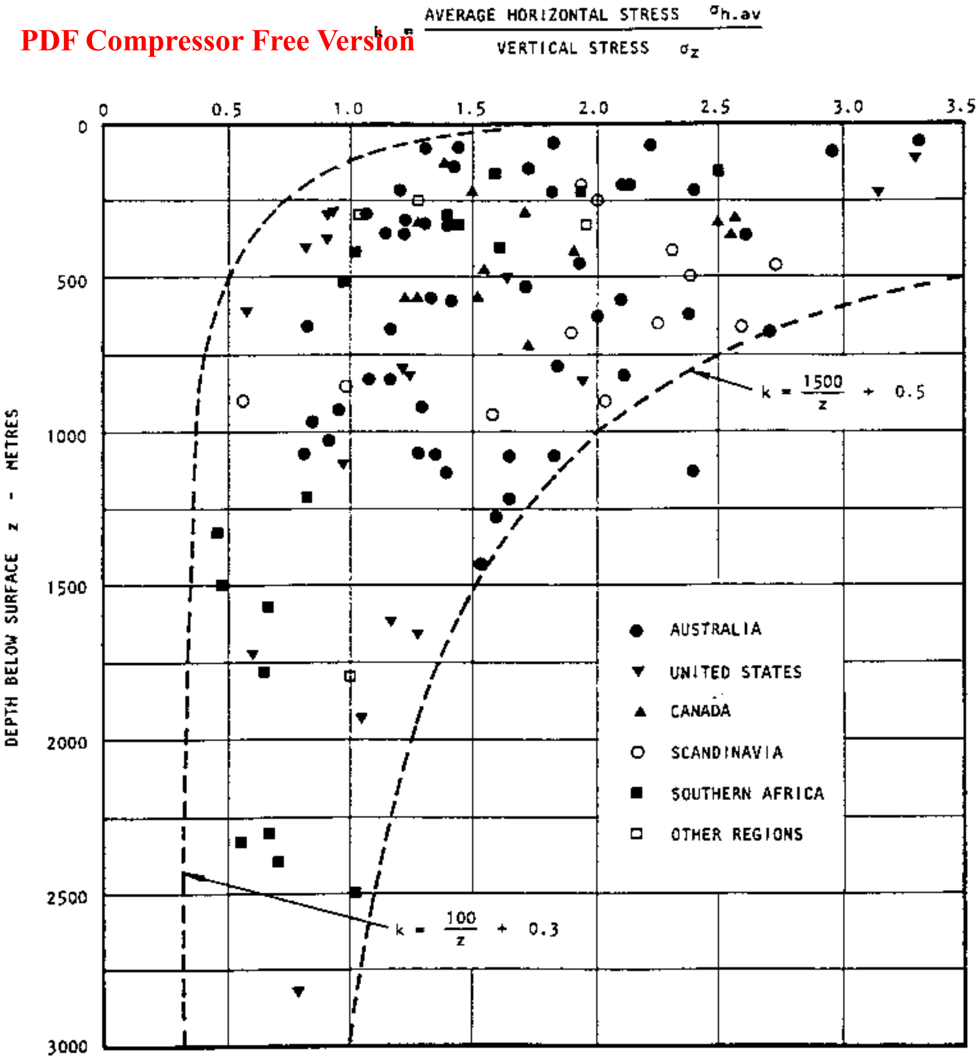


Figure 2.3. Variation in ratio of average horizontal stress to vertical stress with depth below surface (from Brown & Hoek 1978, by permission of Pergamon Press).

expansion, spalling and fretting on unloading and exposure. It is believed that these effects are due partly to the release of stored strain energy (Bjerrum 1967) but also to the absorption of water and subsequent swelling of the clays. (See Section 2.4, Rapid weathering).

2.1.3 *Special effects in valleys*

Gentle anticlines, in some cases with associated thrust faults as shown in Figure 2.8, have been recorded across many river valleys cutting through near-horizontal sedimentary rocks of moderate to low strength. The phenomenon is referred to as 'valley bulging' or 'valley rebound,' and

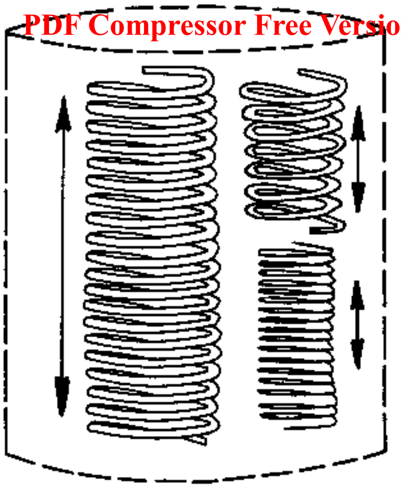


Figure 2.4. Stored strain energy – compressed springs encased in plastic. Analogous to compressed crystals in rocks as they solidify or become cemented (based on Emery 1964).

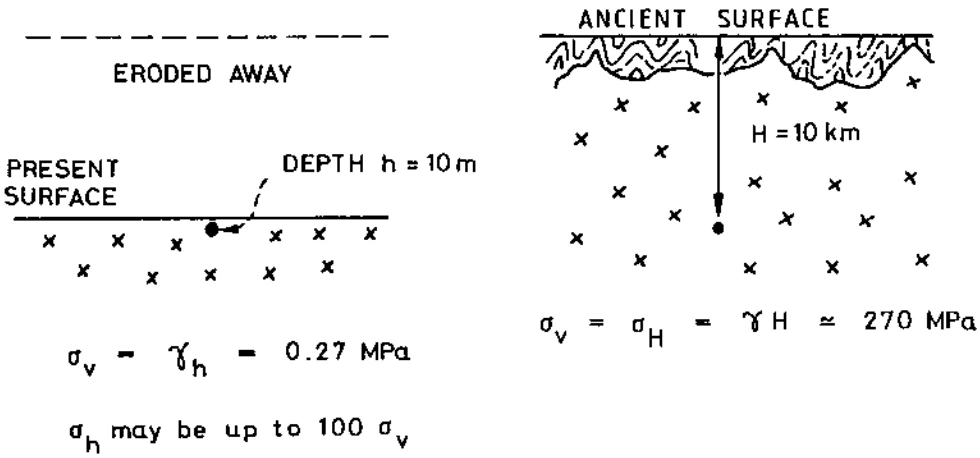


Figure 2.5. One probable cause of high horizontal stresses at shallow depths.

has been described by Zaruba (1956), Simmons (1966), Ferguson (1967), Patton & Hendron (1972), Matheson & Thompson (1973), Horswill & Horton (1976) and McNally (1981). Most of the features shown on Figure 2.8 have clearly developed as a result of buckling and shear failure under high horizontal compressive stresses. The stresses were concentrated beneath the valley floor as a result of load transfer as the excavation of the valley removed lateral support from the rock layers above the floor, and vertical load from the rock beneath the floor. The steeply-dipping joints next to the cliff faces probably opened up due to expansion of the rock layers under the influence of horizontal stresses both across and parallel to the valley.

All of the effects shown on Figure 2.8 were present at the site for Mangrove Creek Dam near Gosford, New South Wales. This 80 m high concrete faced rockfill dam is located in a valley 200 to 300 m deep, cut through an interbedded sequence of sandstones, siltstones and clay-

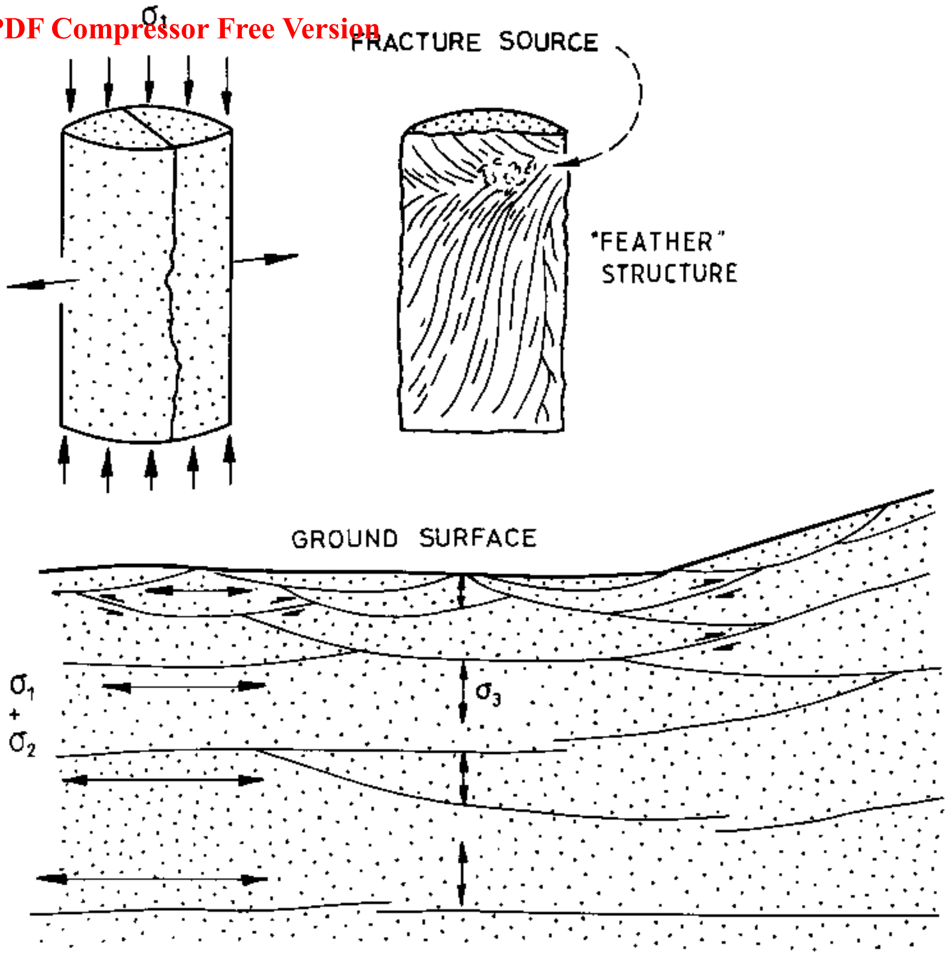


Figure 2.6. Sheet-joints, formed by induced tensile failure (as in an unconfined compression test with zero friction at the platens).

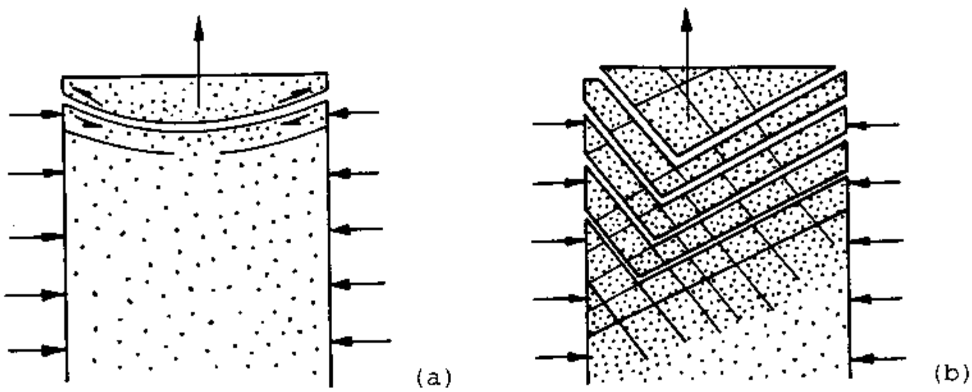


Figure 2.7. Effects of destressing in a) intact rock, and b) jointed rock.

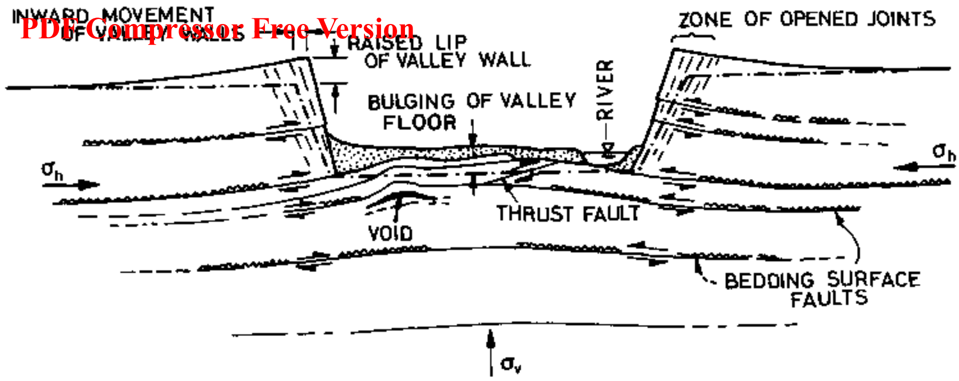
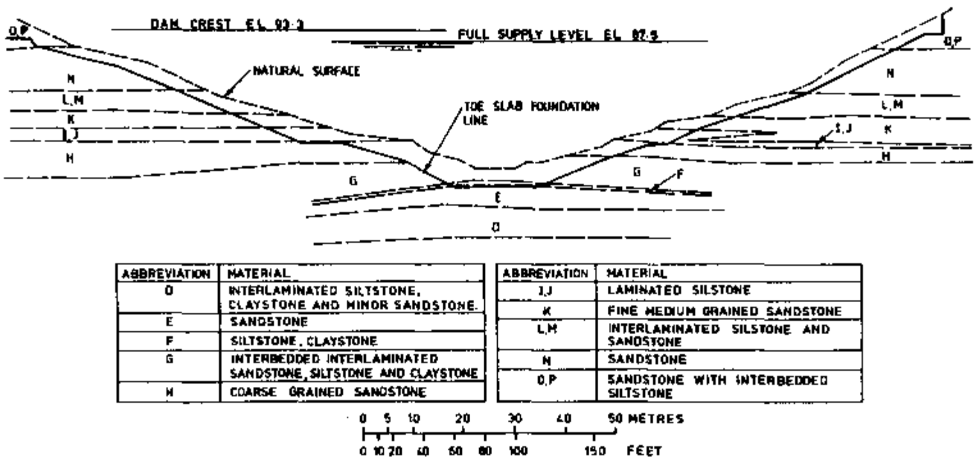


Figure 2.8. Complex valley structures related to stress release in weak, flat-lying rocks (based on Patton & Hendron 1972).



ABBREVIATION	MATERIAL
D	INTERLAMINATED SILTSTONE, CLAYSTONE AND MINOR SANDSTONE.
E	SANDSTONE
F	SILTSTONE, CLAYSTONE
G	INTERBEDDED INTERLAMINATED SANDSTONE, SILTSTONE AND CLAYSTONE
H	COARSE GRAINED SANDSTONE

ABBREVIATION	MATERIAL
I,J	LAMINATED SILTSTONE
K	FINE MEDIUM GRAINED SANDSTONE
L,M	INTERLAMINATED SILSTONE AND SANDSTONE
N	SANDSTONE
O,P	SANDSTONE WITH INTERBEDDED SILTSTONE

Figure 2.9. Cross-section along the grout cap foundation at Mangrove Creek Dam. From MacKenzie & McDonald (1985).

stones (Fig. 2.9). Away from the river the rock layers in the valley sides generally show joints at wide to very wide spacings.

Near and beneath the river bed there is a broad, gentle, 'valley bulge' as shown on Figure 2.9. Although the shape of this feature is not very pronounced, the rock within it was intensely disrupted down to 15 m below the river bed. The sandstone unit E (Figs 2.9, 2.10) showed extension joints which were open or clay-filled, to a maximum of 100 mm, and the underlying unit D (interbedded) contained zones of crushed rock and clay up to 600 mm thick, apparently produced by overthrust faulting (Fig. 2.11). This unit also contained gaping joints parallel to bedding.

To provide a stable foundation for the grout cap and to prevent possible erosion of the clay filling from joints, cable anchors were installed and a concrete diaphragm wall 30 m long and

Figure 2.10. Upper surface of Unit E sandstone at Mangrove Creek Dam valley bulge, showing joints open as much as 100 mm, clay-filled in part.

Figure 2.11. Crushed zone in Unit D (Interbedded siltstone and claystone) formed by overthrusting. Photograph courtesy G. McNally.

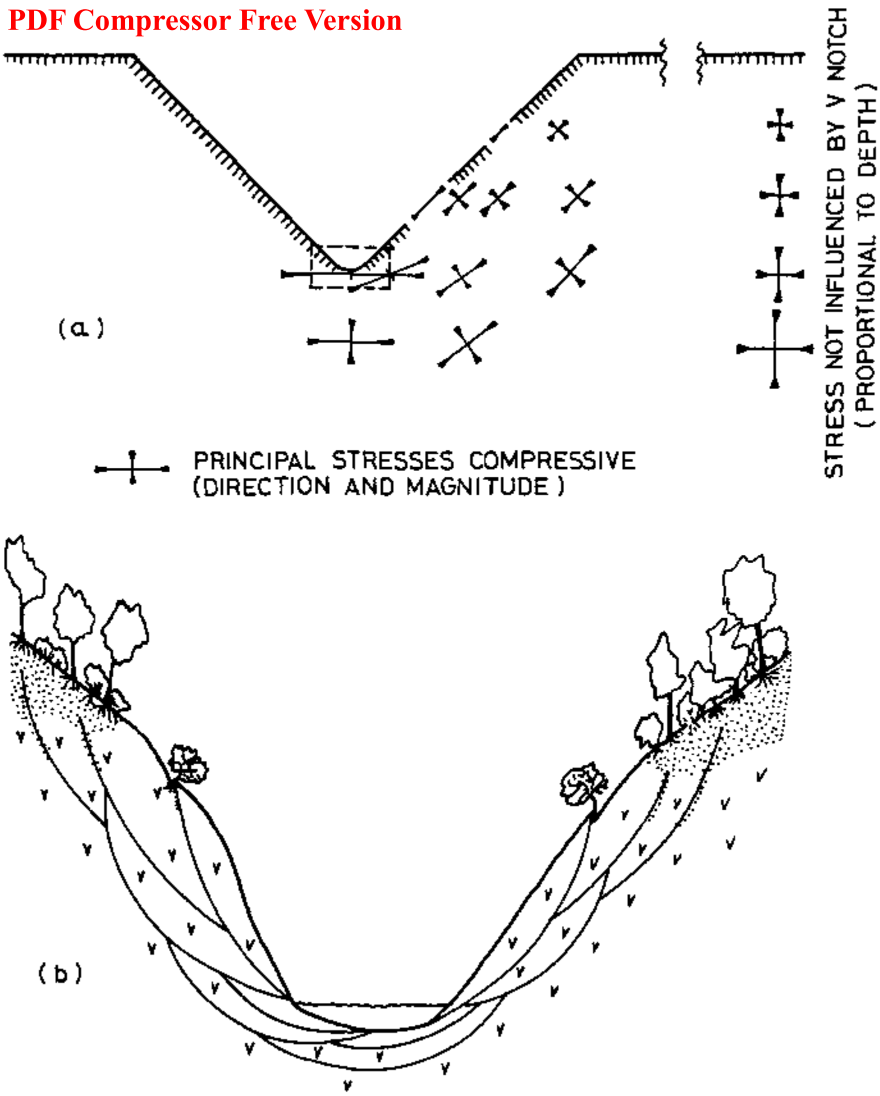


Figure 2.12. High stresses developed in valley floor and lower sides, and resulting pattern of sheet joints in strong rock without 'tectonic' joints (model results from Alexander 1960).

15 m deep was constructed beneath the valley floor (Mackenzie & McDonald 1985).

Although usually not as pronounced as in Figures 2.8 to 2.11, destressing effects are invariably found also at and near the floors of valleys in strong to extremely strong rocks, in areas where high horizontal stresses are known to exist. The effects include sheet joints and gaping or soil-infilled joints of tectonic origin. These joints usually occur both in the valley walls and beneath the floor. It is clear from Figure 2.12a that large stress concentrations are likely to occur in rock beneath the bottom of deep, youthful gorges, and so such destressing effects, even if not visible, are likely to be present, as shown in Figure 2.12b.

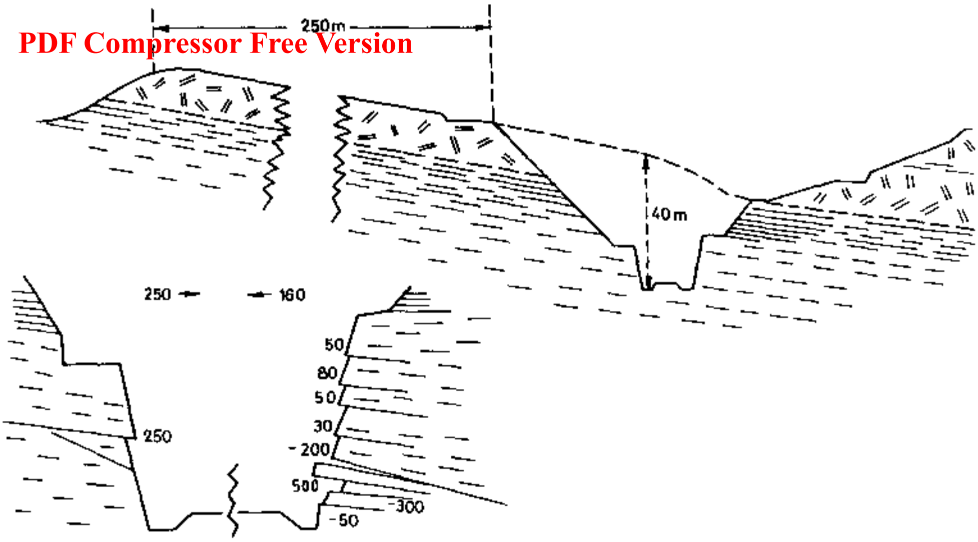


Figure 2.13. Convergence indicated by traces of presplit holes, near base of railway cutting in fresh, strong siltstone.

2.1.4 *Rock movements in excavations*

Spalling, buckling upwards, and the formation of new sheet joints are relatively common occurrences in near-surface excavations in very high strength rocks in North America, Scandinavia and Australia. Emery (1964) reports that an anticlinal fold about 100 m long with crest elevation of almost 5 m was formed overnight in a limestone quarry in Kingston, Ontario. Lee et al. (1979) describe rock bursts in shallow excavations in granite and gneiss in Maine, U.S.A. Ward (1972) describes a 30 m deep excavation in gneiss at New York City, where inward movement of the walls caused rock bolt failures.

Bowling & Woodward (1979) describe 'rockbursts' and the formation of new sheet joints at Copeton Dam in New South Wales. Relatively minor spillway discharges of up to 460 m^3/s caused erosion of a channel about 20 m deep in fresh, massive granite. The granite apparently failed progressively by buckling upwards. As each slab of rock was carried away a new sheet joint formed, by buckling. Isolated rockbursts continued to occur in the channel floor for more than 6 months after the floods. The channel was eventually stabilized by rock anchors and dental concrete.

Inward movement or 'convergence' of the walls of relatively shallow excavations in rock is another common destressing effect. Figure 2.13 shows part of a railway cutting 40 m deep and about 500 m long near Paraburdoo, Western Australia. After excavation of the lowest 15 m of the cutting, by presplitting and trench-blasting methods, a maximum possible convergence of 410 mm was indicated by displacement of traces of presplit holes. Although part of this convergence was undoubtedly caused by the development of blast-initiated cracks behind the rock faces, it is believed that much of the 410 mm was caused by destressing of the gently dipping siltstone. This belief is based on many other similar situations where the authors have observed inward movements to continue for several days after completion of excavations. Similar observations have been reported by Wilson (1970).

2.2 WEATHERING OF ROCKS

Broadly speaking, weathering of a rock is its response to the change from the pressure, temperature, moisture and chemical environments in which it was formed, to its new environment at and near the ground surface. Weathering processes are of two fairly distinct types, namely mechanical, and chemical.

2.2.1 *Mechanical weathering*

Mechanical weathering includes all of the near-surface physical processes which break rock masses down to progressively smaller rigid blocks or fragments, and cause those blocks to separate. Mechanical weathering generally precedes chemical weathering. It renders the rock mass more permeable and facilitates access for groundwaters to large surface areas of rock substance.

Destressing, in particular the formation of sheet joints and the opening up of existing 'tectonic' joints near the ground surface, is the primary and generally the most significant mechanical weathering process (See Figs 2.6, 2.7, 2.8 and 2.12b). The other processes, in order of their (generally) decreasing significance, are as follows:

– gravitational creep (e.g. of slabs and wedges, and toppling),
– joint water thrusting and uplifting during extreme rainfall events,
– earthquake accelerations,
– growth of tree roots in joints,
– expansion of clays in joints,
– freezing of water in joints,
– extreme temperature changes causing differential expansion and contraction of exposed rock faces.

Patton & Hendron (1972) suggested possible mechanical weathering effects beneath the floor and lower sides of valleys in groundwater discharge areas. This type of situation (Fig. 2.14) may be relatively rare, but if it occurs the hydraulic uplift and thrust effects shown are clearly possible.

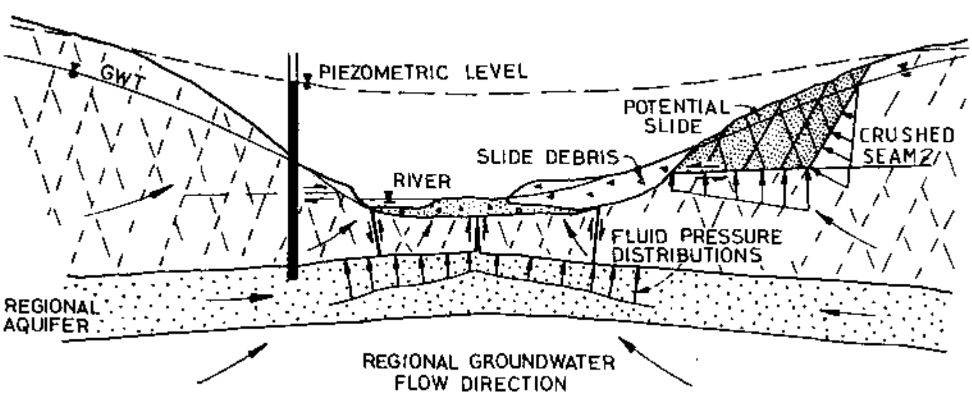


Figure 2.14. Possible effects of high fluid pressures on valleys in groundwater discharge areas (from Patton & Hendron 1972).

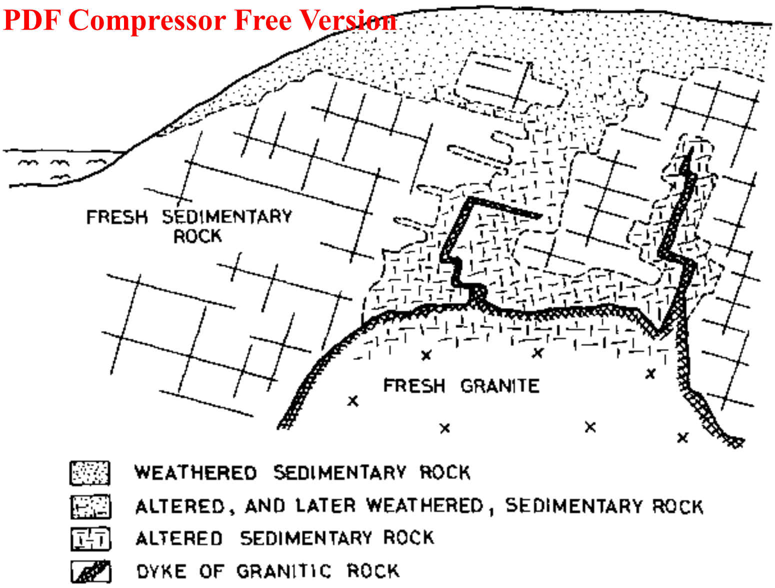


Figure 2.15. Diagrammatic cross section showing hydrothermally altered zone near a granite intrusion, with the uppermost part of the altered zone being weathered subsequently.

2.2.2 *Chemical decomposition*

The term chemical decomposition as used here includes all of the chemical (and to a minor extent physical) processes which cause mineral changes resulting generally in weakening of rock substances, so that eventually they assume soil properties.

Throughout this book, the products of chemical decomposition are described using the terms defined in Section 2.5.1, Tables 2.3 to 2.6.

Chemical decomposition of rocks can be caused by either near-surface (weathering) processes or deep-seated (alteration) processes. Recognition of this distinction is important in civil and mining engineering because the nature and distribution of weathered materials are generally different from those of altered materials (Fig. 2.15).

British Standards Institution (1981) and International Society for Rock Mechanics (1978), differentiate between the terms 'decomposed' and 'disintegrated' for weathered rocks which have assumed soil properties (Table 2.8). They use the former where some or all of the grains are decomposed, and the latter 'where the rock is friable but the mineral grains are not decomposed.' In the opinion of the authors it is not important to distinguish between these two conditions in this way, first because the 'disintegrated' condition as defined is relatively rare, and secondly because in any case the materials (soils) will be described using the Unified Soil Classification (US Department of the Interior 1981) or other.

2.2.3 *Chemical weathering*

Chemical weathering is caused mainly by circulating groundwaters which gain access to low-porosity rock substances via cleavage micro-cracks, open joints and fractures associated

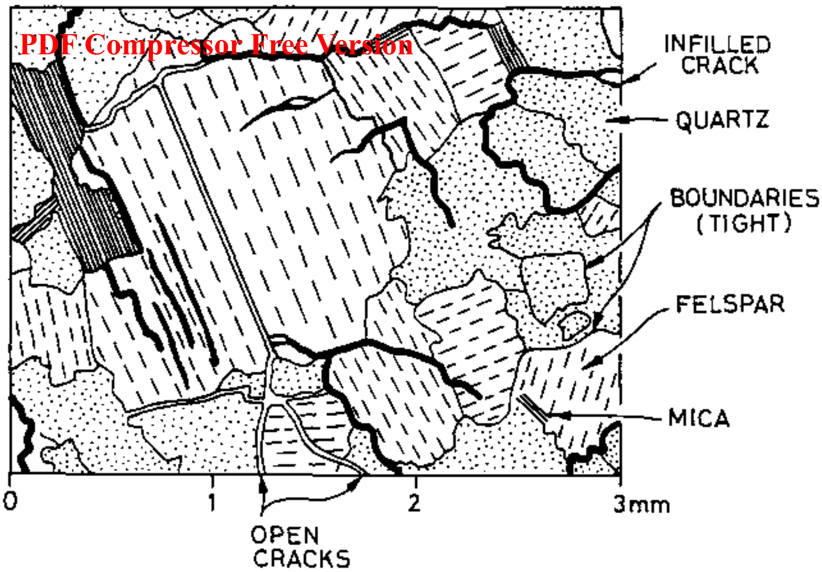


Figure 2.16. Microscopic view of the structure of slightly weathered granite (Dixon 1969).

with faults. In the case of more porous rocks, e.g. some sandstones and limestones, groundwater can also enter through intergranular pores. Most chemical weathering occurs at extremely slow rates, such that the changes to the strength of high strength, non-porous rocks (e.g. granite) are likely to be insignificant during the operating life of most civil engineering projects. There are however some minerals and rocks which decompose, weaken or disintegrate within a few months or years of exposure. These effects will be discussed separately under Section 2.4, Rapid weathering.

Chemical weathering involves the more or less continuous operation of all or most of the following:

1) Chemical reactions between the minerals in the rock, and water, oxygen, carbon dioxide, and organic acids. These reactions cause decomposition of the minerals to form new products, some of which are soluble.

2) Removal of the soluble decomposition products by leaching.

3) Development of microcracks in some rocks, probably due to some decomposition products having larger volumes than the original minerals or to destressing, or capilliary or osmotic suction effects.

4) Deposition of some decomposition products, in pores or microcracks.

Processes 3 and 4 were illustrated by Dixon (1969) who made microscopic studies of granite and schist in a range of weathered conditions. He found that in slightly weathered samples, i.e. rocks which showed only slight discolouration in hand specimen, the first weathering effects visible microscopically were slight discolouration of the felspars and mica minerals, and the presence of many microcracks, some open and others filled with an opaque mineral assumed to be limonite or clay (Fig. 2.16).

Baynes & Dearman (1978a) describe microfabric changes which occur in granites during various stages of weathering, using a scanning electron microscope. In Baynes & Dearman

(1978b) they relate changes in engineering properties, to the microfabric changes.

The chemical reactions involved in chemical weathering include carbonation, hydrolysis, solution, oxidation and reduction. As most cannot be actually observed, details of the reactions, as published by various workers, are in part speculative. A useful account is given on page 23 to 29 of Selby (1982).

2.2.3.1 *Susceptibility of common minerals to chemical weathering*

As would be expected, the susceptibility to weathering (or the 'weatherability') of minerals in igneous rocks varies in accordance with the temperatures at which they were formed. This is illustrated in Table 2.1. With the exception of quartz, the most stable mineral, all of the others on

Table 2.1. Susceptibility of igneous rock-forming minerals to weathering.

Temperature of formation	Susceptibility to weathering	Mineral	Common igneous rock types
Highest	Highest	Olivine	Basalt, dolerite, gabbro
		Calcic felspar	
		Augite	Andesite, diorite
		Hornblende	
		Sodic felspar	
		Biotite	Rhyolite, granite
		Muscovite	
Lowest	Lowest	Quartz	

Table 2.2. Susceptibility of other common minerals to weathering.

Group	Mineral	Effects of weathering
Carbonates	Calcite	Readily soluble in acidic waters
	Dolomite	Soluble in acidic waters
Evaporites	Gypsum	Highly soluble
	Anhydrite	Highly soluble
	Halite (common salt)	Highly soluble
Sulphides	Pyrite and various other pyritic minerals	Weather readily to form sulphates, sulphuric acid and limonite
Clay minerals	Chlorite	Weathers readily to other clay minerals and limonite
	Vermiculite	Weathers to kaolinite or montmorillonite*
	Illite	Weathers to kaolinite or montmorillonite*
	Montmorillonite	Weathers to kaolinite
	Kaolinite	Stable**
Oxides	Haematite	Weathers to limonite
	Ilmenite	Stable
	Limonite	Stable

*These minerals expand and contract with wetting and drying and this can cause large disruptive forces and disintegration of some rocks.
**Softens on wetting.

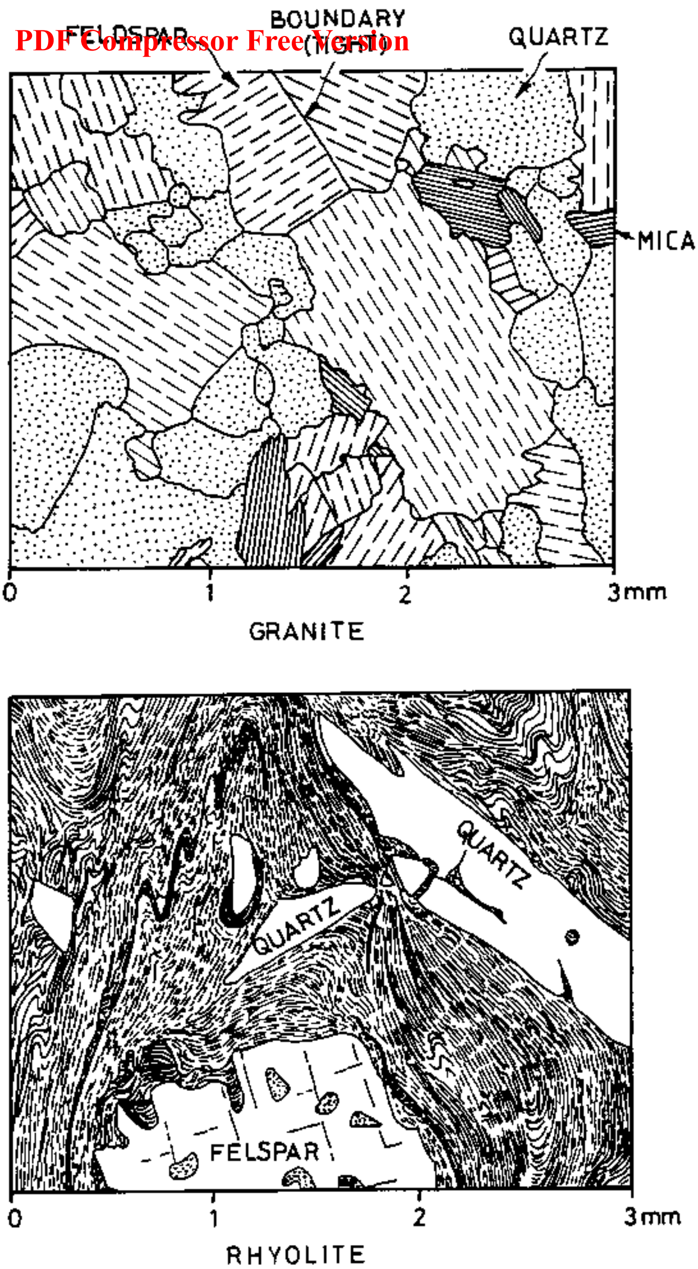


Figure 2.17. Microscopic views of the structure of a) granite and b) rhyolite.

Table 2.1 weather eventually to clay minerals. Quartz is slightly soluble in water. It is almost unaffected by weathering except under tropical conditions when it is readily dissolved (in a geological time-frame).

The susceptibility of other common minerals to weathering is indicated in Table 2.2. The minerals in the carbonate and evaporite groups (shown in this table) may occur in any of three different ways, namely:

– as rocks, e.g. calcite as limestone, or dolomite as dolomite (rock) or dolostone, or

– as cements in sedimentary rocks, e.g. calcite as cement in sandstone composed mainly of quartz grains, or

– as veins or joint fillings or coatings in any rock mass.

2.2.3.2 *Susceptibility of rock substances to chemical weathering*

The susceptibility of a rock substance to weathering depends upon the following:

– the susceptibility to weathering of its component minerals, and

– the nature of its fabric, i.e. the degree of interlock and/or cementation of the mineral grains,and

– its porosity and permeability.

For example, both rhyolite and granite contain sodic felspar, micas and quartz, but in rhyolite the crystals are much more fine grained and more tightly interlocked than in granite. Hence granite is generally more susceptible to weathering than rhyolite (Fig. 2.17).

Also, a dense, non-porous limestone comprising almost 100 percent calcite is likely to be less susceptible to weathering (solution) than a porous sandstone comprising 80 percent quartz grains which are durable but cemented by calcite. (See also Chapter 3, Section 3.7.2).

2.2.4 *Weathered rock profiles and their development*

The following are the main factors which contribute to the development of weathered profiles:

– climate and vegetation,

– rock substance types,

– defect types and patterns,

– erosion,

– time,

– topography,

– groundwater.

The influence of each of these factors is discussed in the sections which follow. Although the factors are discussed separately or in pairs, they usually all interact and influence the development of weathered profiles. The terms used to describe weathered rocks are defined in Section 2.5.

2.2.4.1 *Climate and vegetation*

Climate is the dominant factor. At one extreme, in a desert situation, chemical weathering effects are usually almost negligible. Mechanical weathering effects may include opening up of joints due to destressing and some fragmentation of exposed rock surfaces due to extreme temperature changes. At the other extreme, under hot, humid (e.g. tropical or sub-tropical) conditions chemical weathering proceeds relatively rapidly, due to the ready availability of water containing oxygen, carbon dioxide and organic acids derived from the vegetation. Regardless of the composition and structure of the parent rock mass, the near-surface weathered profile is usually of the lateritic type, the upper part of which may consist almost entirely of oxides of iron and aluminium. Figure 3.42 shows a typical near-surface profile of this type, and the processes believed to be involved in its development. It must be appreciated that the weathered profile at a particular site may not have been developed under the present climatic conditions. Throughout some arid areas of Australia and South Africa there are deep weathered profiles which developed under tropical or semi tropical conditions, largely during Tertiary time.

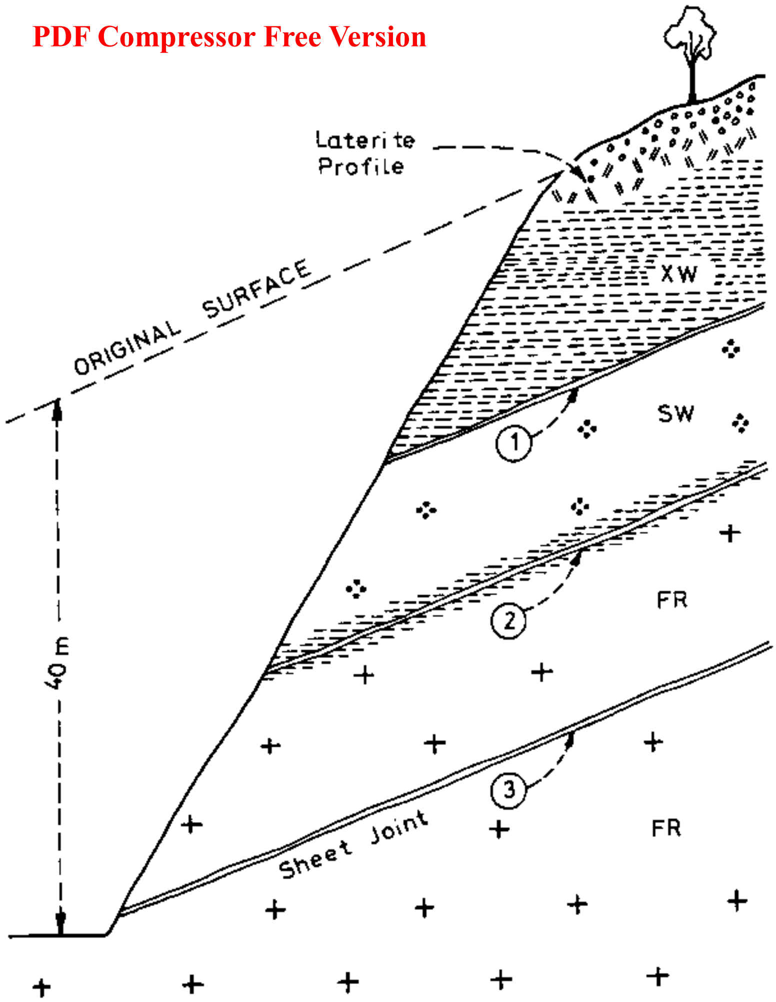


Figure 2.18. Weathered profile developed previously on sheet-jointed gneiss, Darling Range, Western Australia (from Gordon 1966).

2.2.4.2 *Rock substance types, and defect types and pattern*

Many fresh rock masses are relatively complex in their composition and structure. They may contain several different rock types, with widely differing substance strengths and susceptibilities to chemical weathering, and the rocks may be folded and intersected by defects such as joints and faults. The material in the fault zones may be crushed rock which has essentially 'soil' properties in the fresh (unweathered) state. Chemical weathering generally proceeds from the joints and faults, which act as groundwater conduits. Because of this, the distribution of intensely weathered rock is usually governed as much or more by the pattern of occurrence of these defects, than by the depth below the ground surface. Figures 2.18 to 2.25 show a range of weathered rock profiles, illustrating these effects.

Figure 2.18 is a profile developed under tropical conditions in a gneiss rock mass with very

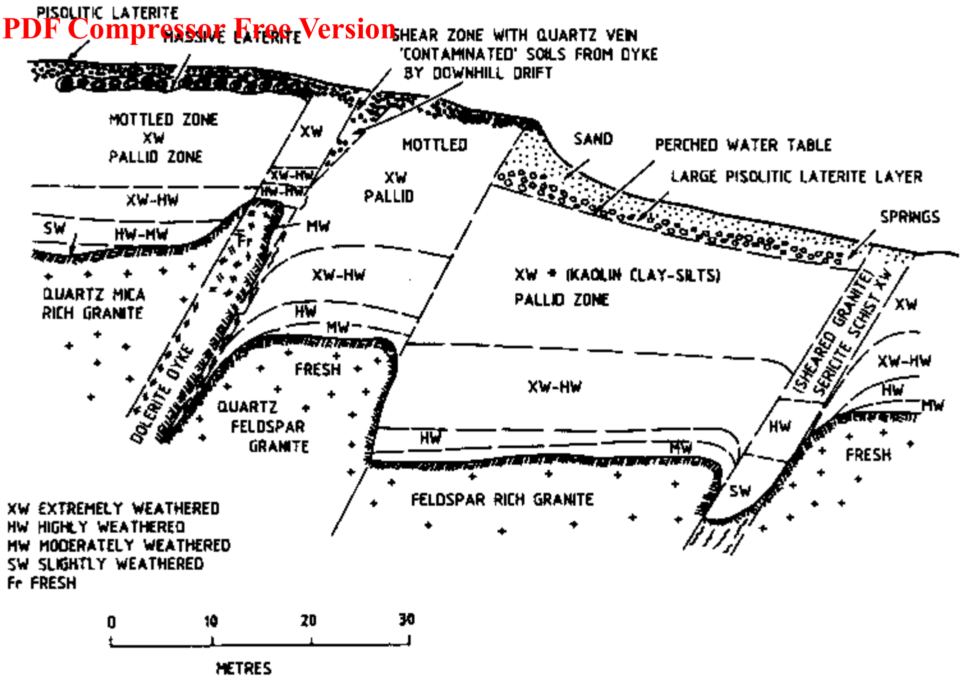


Figure 2.19. Diagrammatic cross section showing weathered profile controlled partly by rock type and partly by sheared zones, Darling Ranges, Western Australia (from Gordon 1984).

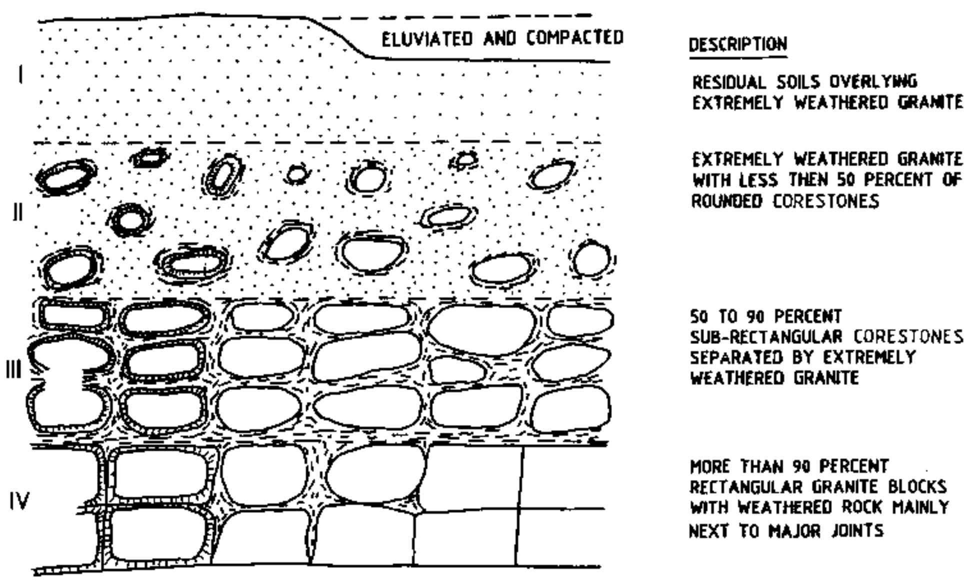


Figure 2.20. Idealised weathered profile in granitic rocks. From Ruxton & Berry (1957).

simplest structure – sheet joints parallel to the original (sloping) ground surface. The uppermost 13 m comprises a laterite soil profile overlying extremely weathered gneiss. The extremely weathered gneiss is a gravelly clay (CL-GC). Its upper boundary with the pallid horizon of the laterite profile is gradational, but its lower boundary with slightly weathered gneiss is sharp, coinciding with Sheet Joint (1). It is likely that several other sheet joints were present initially at shallower depth, but all trace of them has been obscured by the extreme weathering or by disturbance of the exposed surface of the cut. Between Sheet Joints (1) and (2) the gneiss is slightly weathered except for a narrow extremely weathered zone which surrounds Sheet Joint (2). Below this zone the gneiss is fresh. It is clear from this simple profile that chemical weathering proceeded both from the ground surface and from the sheet joints.

Figure 2.19 shows a more complex profile developed under tropical conditions in the same region as the profile in Figure 2.18. The profile steps downwards (i.e. deepens) to the east because of the presence of three different types of granite which are progressively less siliceous and hence are more susceptible to chemical weathering. The dolerite dyke has a high resistance to weathering and so forms a prominent ridge in the upper surface of the dominantly fresh rock zone. The eastern contact between the dyke and granite is sheared, and weathering of the

Figure 2.21. Granitic boulders or corestones showing spheroidal weathering effects. Photo courtesy of Dr R.Twidale.

sheared rock has resulted in a deep, narrow slot of highly to extremely weathered rock. A local depression in the fresh rock zone occurs also along a sheared zone near the eastern side.

Figure 2.20 shows an idealised weathered profile through granitic rocks in Hong Kong described by Ruxton & Berry (1957). Broadly similar profiles are found in many other areas underlain by granitic and other igneous rocks, and this profile has been widely accepted as typical for such rocks. It is clear that the main controls on the distribution of weathered materials have been the depth below the ground surface and the pattern of the joints in the rock mass.

The corestones shown on Figure 2.20 usually display spheroidal weathering effects, that is, they comprise fresh or slightly weathered rock surrounded by concentric shells of rock which becomes progressively more weathered away from the core, as shown on Figure 2.21. The cracks which isolate the shells are like small scale sheet joints and are probably caused by relief of residual stresses and other stresses set up by capillary, osmotic and other chemical weathering processes.

Figure 2.22 shows what appears to be an excellent example of the idealised weathered granitic profile of Ruxton & Berry (1957), exposed in elevation view, by erosion. The upper slopes show mainly extremely weathered granite (soil properties). In the lower, rocky slope this material occurs between corestones and becomes progressively less abundant, and is eventually absent in the outcrops close to water level.

The actual situation on this hillside is not so simple. Near the centre of the photograph, on the skyline and elsewhere at intermediate levels on the slope, hidden by trees, there are granite boulders and areas of what appear to be outcrops of essentially fresh granite. Excavations made into slopes showing similar surface evidence have shown subsurface profiles as on Figure 2.23.

At its right hand edge this figure shows a weathered profile similar to that of Ruxton & Berry

Figure 2.22. Granite corestones and outcrops below granitic soils on Granite Island, Victor Harbor, South Australia.

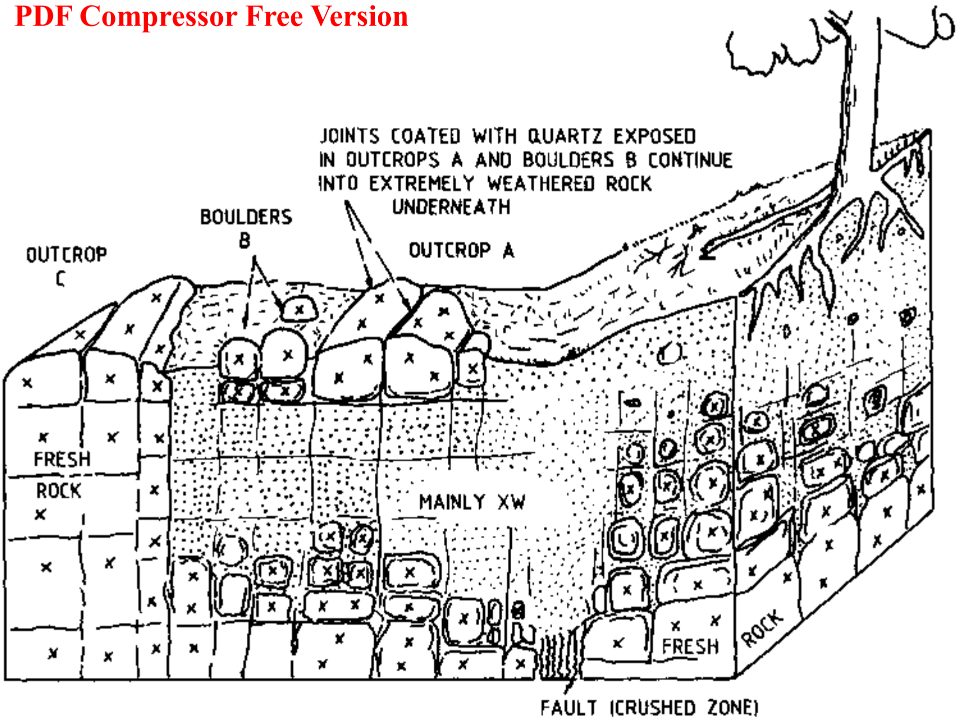


Figure 2.23. Features sometimes seen in weathered masses of granitic and other igneous rocks.

(1957) except that the extremely weathered material extends locally much deeper along and next to the fault. Near the centre the profile at depth is also similar to that of Ruxton & Berry (1957). However, outcrop area A and the boulders B occur at the ground surface and are underlain by 7 to 10 m of extremely weathered material. On the left side, outcrop area C looks similar to area A but is continuous downwards into mainly fresh bedrock.

The differences between the profiles A, B and C raise the following questions:

– Are outcrop A and boulders B really *in situ*?

– If they are *in situ*, then why don't they continue downwards into progressively less weathered rock mass as is the case at outcrop area C?

It can usually be shown from the continuity of the joint and fault pattern within the extremely weathered rock, whether or not masses of rock such as A and B are *in situ*. In Figure 2.23 the continuity of coated joints is clear evidence that both are *in situ*.

It is often not possible to provide an unequivocal answer to the second question. One possibility is that the extremely weathered rock under A and B was partly decomposed by chemical alteration before becoming weathered. Another is that the rock substance forming A and B may be for some reason more resistant to chemical weathering than that which occurred initially adjacent to and below them. Yet another is simply that the joints which surrounded these rock masses were less permeable than those elsewhere, allowing much less groundwater percolation.

Although it is not often possible to understand the reason for such anomalies in weathered

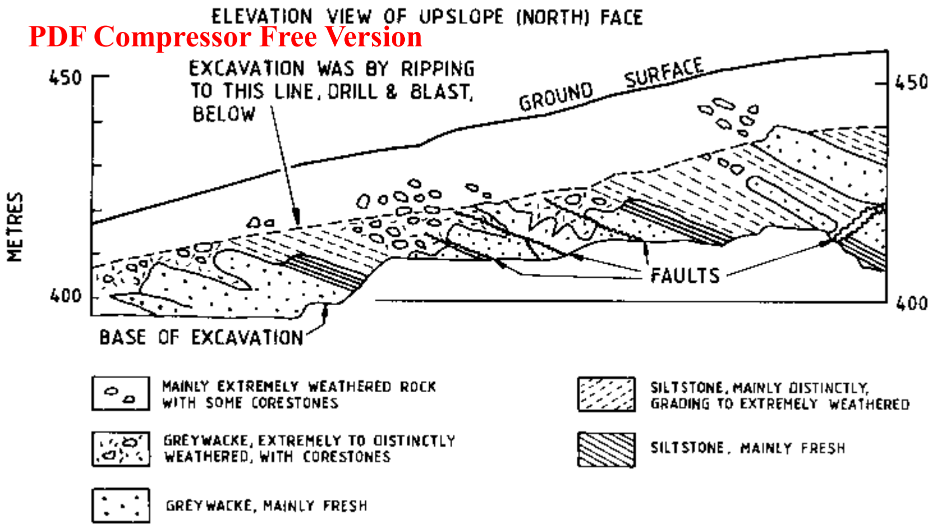


Figure 2.24. Distribution of fresh and variably weathered rock, controlled largely by variation in rock substance type, at Split Rock Dam, New South Wales.

profiles in apparently 'uniform' igneous rocks, it is very important to appreciate that they occur quite commonly, and hence to expect them and allow for them in site investigations.

Figure 2.24 shows the nature of part of the weathered profile developed at the spillway of Split Rock Dam, New South Wales. The spillway excavation below the depth of ripping refusal was the designated quarry to supply rockfill for the dam. Alternating beds of greywacke, greywacke breccia and siltstone dip gently into the hillslope. The greywacke is extremely strong when fresh and relatively resistant to chemical weathering. The siltstone is strong to very strong when fresh, but is more susceptible to chemical weathering than the other rocks. The resulting profile, with beds of fresh greywacke (extremely strong) overlying variably weathered siltstone (weak rock grading to soil properties) made it difficult to meet some rockfill requirements, with normal quarrying methods.

2.2.4.3 *Time and erosion*

The depth of a weathered profile depends upon the amount of time during which the rocks have been exposed. However, this factor (time) must be considered together with erosion, because erosion will have been continually lowering the ground surface, during the development of the profile. In tectonically stable areas where the effects of erosion have been relatively small, great depths of weathered rock have developed. For example, large areas of Western Australia which have been relatively stable and mainly exposed since Precambrian time, are now almost flat 'peneplains,' underlain by extremely weathered rock to depths of 30 to 50 m. In tectonically active areas uplift and subsequent erosion occur, and result in much shallower weathered profiles in comparable rocks. In the areas of North America and Scandinavia eroded by ice-sheets during the Pleistocene glaciation, there is generally no chemically weathered profile.

Figure 2.25 shows a situation which is frequently seen: weathered profiles which are deepest beneath hilltops, ridges and plateaus, and shallow or absent at or close to the floors of valleys. This situation may have arisen simply because the rate of valley erosion (i.e. removal of

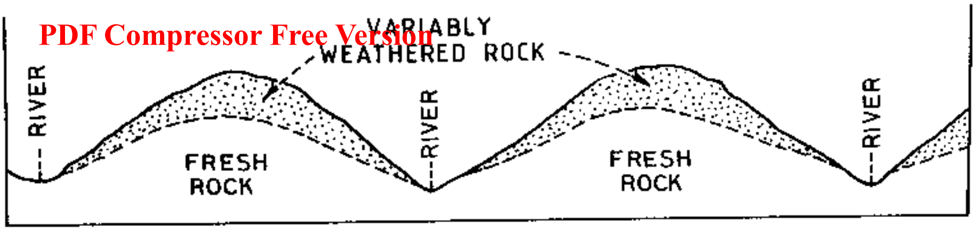


Figure 2.25. Shape of upper surface of fresh rock in a situation where the rate of valley erosion has been exceeding the rate of lowering of the chemically weathered zone.

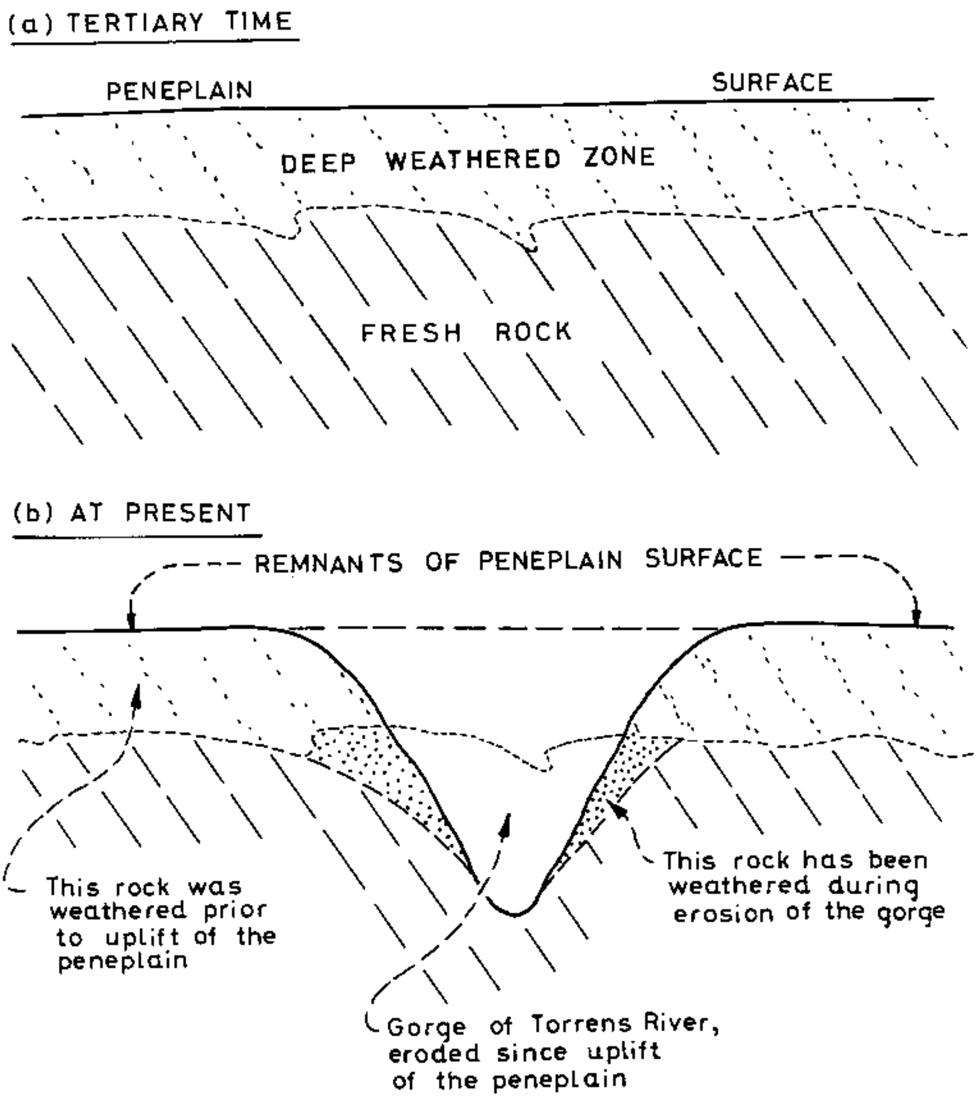


Figure 2.26. Type of valley weathered profile where a river has cut down through an uplifted block which has been deeply weathered previously.

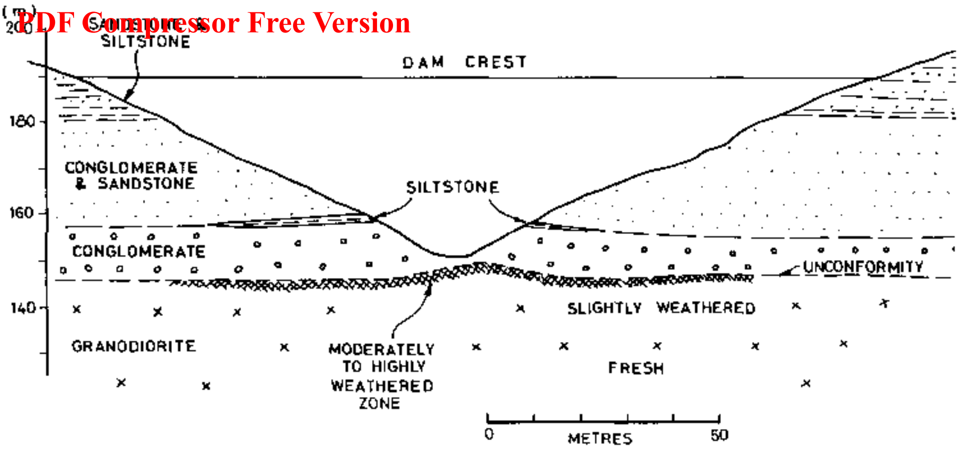


Figure 2.27. Cross section at Yellow Pinch Dam, New South Wales, showing buried land surface with weathered zone.

weathered materials) has exceeded the rate of deepening of the weathered profile. However, in some places situations similar to that in Figure 2.25 can have a different origin as indicated in Figure 2.26. This shows the situation in parts of southern and eastern Australia, where mountain ranges were formed during late Tertiary time, by the faulting and uplifting of deeply weathered rock. Relatively rapid erosion of river valleys since late Tertiary time has resulted in composite weathered profiles as shown on Figure 2.26b. The rock near the valley floors is mainly fresh but is usually somewhat mechanically loosened due to destressing.

Ancient weathered profiles can be buried and preserved beneath younger sediments or even sedimentary rocks. Figure 2.27 is a diagrammatic cross section showing the geological situation at Yellow Pinch dam in New South Wales. The valley walls at this site are formed by a near-horizontal sequence of mainly conglomerates and sandstones, of Permian age. These rocks are slightly weathered. However, at and just below river level, they unconformably overlie granodiorite of Devonian age. The granodiorite is moderately to highly weathered in its uppermost 2 to 5 m. The upper surface of the granodiorite is an eroded and weathered land surface of Permian age.

2.2.4.4 *Groundwater and topography*

Optimum conditions for chemical weathering occur where vertical movement of aerated groundwater is at a maximum, i.e. beneath elevated flat ground or gentle slopes, above the main water table. Beneath steep slopes there is less infiltration of surface waters due to rapid runoff.

Field evidence in many places indicates that very little weathering has occurred beneath the main or fundamental water table. This is probably due to less oxygen and slower groundwater movement, below the water surface. In some places it is due to the fact that the main water table is in fact perched above fresh rock which is effectively impervious. However, where suitably permeable rocks occur beneath the main water table, rapid groundwater flow can occur well below the water table, and chemical weathering (or solution, in soluble rocks) occurs.

It must be appreciated that due to changes in climate, the present water table may be higher or lower than the water table which dominated when a particular weathered profile was formed.

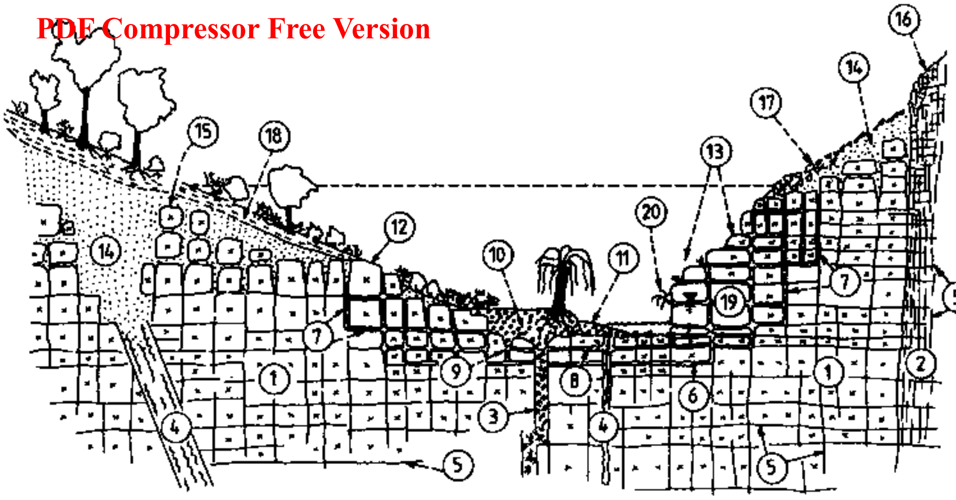


Figure 2.28. Diagrammatic cross section through the floor and lowest slopes of a steep-sided river valley, showing the types of feature developed by mechanical and chemical weathering processes.

2.2.4.5 *Features of weathered profiles near valley floors*

Figure 2.28 is a hypothetical cross section through the lower part of a relatively steep valley in relatively strong, jointed rocks. It shows diagrammatically a number of features found commonly when excavating foundations for dams.

The river is flowing in an entrenched meander channel and at this site it has been migrating towards the right bank. As a result of this the left bank slopes are flatter, and show less rock outcrop, than the right bank slopes. The river channel occupies only half of the valley floor, the left side being formed by a well established (overgrown) alluvial terrace.

There are three bedrock types: granite (1) and microgranite (2) which are fresh and very strong at depth, and basalt (3) in the form of an intrusive dyke which is distinctly altered, weak. The granite contains two faults (4) in which the rock is sheared and partly crushed to soil properties. Joints at depth (5) are mainly tightly closed, but as the valley floor is approached they are either slightly open (6) due to destressing, or else (7) have been open but are now partly or wholly infilled with clay which has migrated down from the chemically weathered zone. In the bedrock just below the river, some joints (8) have become infilled with alluvial silt which has migrated down from the river.

It can be seen that although the valley in profile is V-shaped, the profile of the base of the main mechanically weathered zone is closer to U-shaped. This is often found to be the case, and is due to pronounced destressing at the highly stressed base of the V-notch (see Fig. 2.12a). In this very strong rock there is no obvious bulge or antiform structure across the valley floor, but the opened joints beneath and next to the river bed (including that part beneath the alluvial terrace) indicate that significant rebound movements have occurred.

The actual river bed below the alluvium (10) and (11) contains a slot (9), eroded differentially by the river, along an infilled joint. Similar eroded slots occur over the altered dyke (3) and fault (4).

Upslope from the river there are small granite outcrops (12) on the left bank and larger and steeper granite outcrops (13) on the right bank.

Above the levels of these outcrops the granite is variably weathered. Extremely ranging to distinctly weathered rock (14) locally contains corestones (15) of fresh and slightly weathered rock, and extends downwards next to some joints in the dominantly fresh rock. Extremely weathered rock extends down to about river level, along the fault zone (4).

The microgranite is very resistant to chemical weathering, and forms fresh and slightly weathered steep outcrops (16) at the ground surface. This rock has closely spaced joints, however, and is mechanically weathered near the surface. Toppled and fallen blocks of micro-granite have formed the scree (17) which covers the slope below.

On the left bank, slopewash soil (18) derived from extremely weathered granite occurs to shallow depth beneath the ground surface in most places, becoming deeper locally over the fault (4) and beneath gullies.

The water table (19) on the right bank, daylights at a seeping, partly infilled joint. Ferns and swampy type grasses (20) are established along the outcrop of this joint.

The alluvial gravels (10) beneath the terrace are very old (of Pleistocene age, probably). Gravels of this age are commonly partly weathered. Sands and gravels of the present day point bar (11) are not weathered – they are deposited during the dying stages of present-day floods.

2.2.5 *Complications due to cementation*

It can be seen from all of the above that weathered rock profiles can be highly variable and complex, due to the interaction of both dependant and independant variables. However, weathered profiles can be even more complex and difficult to define because of the following.

2.2.5.1 *Rock cementation*

Chemical weathering does not always weaken rock substances. Some sandstones, e.g. the Hawkesbury and Narrabeen Sandstones in New South Wales are in some places strengthened during weathering, by the deposition of limonite in intergranular pores. These rocks are quartz sandstones, and the matrix between the quartz grains in the fresh rock is usually mainly clay with minor amounts of silica. The limonite is a much stronger cement. Siltstones and sand-stones of the Dargile Formation in Victoria, Australia are also sometimes found to have been locally strengthened by limonite cementation, in the weathered profile.

Local strengthening due to cementation is common also in weathered carbonate-rich rocks – calcite dissolved from one place has been deposited in another.

2.2.5.2 *Rock defect cementation*

Rock defects also are not always made weaker by chemical weathering. Joints which have been infilled and strongly cemented by limonite are relatively common in the weathered zone in the Victorian siltstones mentioned above. It is not uncommon to find that weathering has resulted in strenthening of defects due to the deposition of cements in them, while the adjacent rock substance has been weakened by weathering.

2.3 CHEMICAL ALTERATION

As discussed already in Section 2.2, some igneous rocks have been partly or wholly decom-posed by hot waters and gases during late stages of their solidification. The hot waters may be derived from the igneous magma or else they may be groundwaters which have penetrated to great depth and become heated by the magma. Sedimentary and metamorphic rocks in the vicinity may also become partly or wholly decomposed (Fig. 2.15).

This type of deep-seated decomposition is called hydrothermal alteration. It involves the chemical breakdown of some of the primary (original) minerals to form secondary minerals, which are generally weaker and less stable in water, than the primary minerals. The secondary minerals include serpentine and montmorillonite (from olivine), chlorite (from biotite) and kaolin (from felspars). The altered rocks are therefore generally weaker, and more susceptible to chemical weathering, than the same rocks when unaltered.

Altered rocks can often be distinguished from weathered rocks by their colour – usually green or white, whereas weathered rocks are usually yellow-brown or brown due to oxidation which has produced limonite in most of them. It is important in dam engineering to be able to distinguish between altered and weathered rocks. This is because in an altered rock situation the altered materials may be overlain by fresh rock, and become progressively more altered and weaker, with increasing depth. It is possible to find both altered and weathered rock at the same site, with some of the weathered rock being previously altered (See 2.2.4.2 and Fig. 2.23).

2.4 RAPID WEATHERING

Rapid weathering processes are defined here as those processes which cause exposed rocks to be significantly weakened during periods of only days, months or years. In general the rocks most affected by rapid weathering are relatively porous and not of high strength. However even some very high strength rocks can be affected and so careful observational studies and testing are needed whenever rocks are to be used as construction materials.

2.4.1 *Slaking of mudrocks*

The weakest claystones and shales, which can barely be described as rock, have usually been strengthened only by consolidation. When such 'compaction shales' are exposed in excavations they usually develop fine cracks at the exposed surface due to destressing and drying out. Swelling, further cracking and sometimes complete disintegration back to clay occurs, when the destressed shale or claystone is allowed to absorb water.

Stronger claystones, siltstones and shales (i.e. in the weak to strong rock range) have usually received some strength by cementation and/or recrystallisation of clay minerals to form micas, as well as by consolidation. These rocks also generally develop fine drying out cracks when exposed by excavation but the cracking is much less severe and occurs more slowly than in the 'compaction shales.' Once the fine cracks appear on the rock surface, addition of water causes relatively rapid deterioration of the rock. The reasons for this are believed to be as follows. Water is absorbed rapidly into the cracks and adjacent rock substances, by capilliary suction. The water compresses the air trapped in the cracks and allows clay minerals in the adjacent rock to swell slightly. As a result the cracks widen and propagate.

Most strong siltstones develop cracks eventually with repeated wetting and drying. However in constant humidity environments such as in rockfills or earthfills, such rocks have been found to have suffered little or no breakdown over periods of up to 90 years.

The slaking of mudrocks and its significance in dam construction is discussed further in Chapter 3, Section 3.5.5.

Some poorly cemented sandstones which contain a high proportion of clay minerals in their matrices, show similar slaking properties to mudrocks.

2.4.2 *Rapid solution*

Any rocks (usually sandstones or siltstones) which contain appreciable amounts of evaporite minerals (gypsum, anhydrite or halite) as cement or matrix, are likely to be weakened rapidly by solution of these minerals when inundated or when exposed to repeated wetting and drying. Rapid solution of gypsum cement in a conglomerate was suggested as a contributing factor to the failure of St. Francis dam in California in 1928 (Ransome 1928, Clements 1972).

James & Lupton (1978) and James & Kirkpatrick (1980) provide predictions on solution effects if dams are constructed on foundations containing these minerals. Their studies and predictions are discussed in Chapter 3, Section 3.7.

Highly porous, low density limestones composed of cemented shell fragments are likely to be dissolved relatively rapidly if fresh acidic water is allowed to flow through them. The likely solution mechanisms and rates as predicted by James & Kirkpatrick (1980), are presented in Chapter 3, Section 3.7.4.

2.4.3 *Crystal growth in pores*

Where rock is exposed in saline environments, e.g. as rockfill or riprap on marine breakwaters and some tailings dams, some salt is absorbed by the rock either by periodic inundation or from spray. During warm, dry conditions the rock dries out and salt crystals grow in pores near its surface, disrupting mineral grains (as in the Sodium Sulphate Soundness Test). Periodic wetting up with rainwater causes further disruption. It appears that fresh water is drawn rapidly into the pores by osmotic and capilliary suction, compressing entrained air. The amount and rate of degradation which occurs by the above process depends largely upon the porosity, texture and strength of the rock.

2.4.4 *Decomposition of sulphide minerals*

Trudinger (1973) describes the rapid fretting of surfaces of quartz-sericite schist exposed in excavations at Kangaroo Creek Dam, South Australia. The affected schist was found to contain small amounts (up to 5 percent locally, but generally much less than 1 percent) of metallic sulphide minerals (mainly pyrite and chalcopyrite). Absorption of moisture and oxygen on exposure causes oxidation of the sulphides to form metallic sulphates, and sulphuric acid which attacks other minerals in the rock to form other sulphates. Growth of sulphate crystals in pores within 5 mm of the rock surface causes disintegration and fretting of this near-surface layer.

Trudinger examined schist blocks at 1 to 2 m depth in 40 year old rockfill near Kangaroo Creek Dam, and found no sign of deterioration of these blocks, all of which contained pyrite. He concluded that the fretting is a direct result of the absorption and evaporation of moisture, and hence it does not occur in the constant moisture environment at depth in a rockfill.

2.4.5 *Surface fretting due to electro-static moisture absorption*

Within 3 years of the completion of Kangaroo Creek Dam, it was observed that fretting was occurring on the undersides of some large schist blocks exposed on the downstream face. Many of these blocks did not appear to contain sulphide minerals, and the fretted flakes did not have the characteristic taste of sulphate salts.

West (1978) confirmed that much of the deterioration of the exposed schist blocks was not caused by the sulphide/sulphate effects described by Trudinger. He demonstrated that the likely cause was cyclic adsorption-desorption by the near-surface few millimetres of the schist, of moisture from the air, in a semi-confined or sheltered environment. The moisture changes resulted in expansion and contraction, probably of the sericite, in this near-surface layer.

2.4.6 *Expansion of secondary minerals*

Some rocks appear fresh and strong but in fact have been chemically altered (see Section 2.3) and contain highly expansive secondary minerals. If these are in sufficient quantities they can cause the rocks to be significantly weakened or even to disintegrate, on inundation, or on exposure to the weather. The rocks most commonly affected are basic igneous rocks (basalt, dolerite and gabbro), and the most common expansive secondary mineral is montmorillonite (see also Chapter 3, Section 3.2.3).

2.5 CLASSIFICATION OF WEATHERED ROCK

From the discussion in Sections 2.2 to 2.4 it is not surprising that weathered rock masses are usually complex and variable. 'Ideal' weathering profiles such as those described by Ruxton & Berry (1957) (Fig. 2.20) and Deere & Patton (1971) are useful to illustrate principles and for general description of whole weathered profiles, but the authors consider that the primary system for description of weathered rock should be more basic. It should provide descriptive terms with which the site investigator can record in detail the distribution of weathering products during routine mapping and logging. This distribution can then be used to provide an understanding of the local (site) relationships between the intensity of weathering and the common controlling factors discussed in Sections 2.2 to 2.4, in particular:

– The distribution of rock substance types.
– The pattern of defects in the mass.

Such understanding should assist the site investigator to make soundly based correlations, and predictions about the distribution of rock in various weathered conditions in other parts of the site not directly explored.

It is considered important also that the classification enables the determination of any site relationships which may exist between the strengths of rock substances and their weathered state or condition.

2.5.1 *Recommended classification system*

The classification system for weathered rock substance (Table 2.3), when used in conjunction with the ISRM substance strength classification (Table 2.4) has been shown to meet these objectives.

Using this approach, rock substances are classified as in the examples in Table 2.5. Note that altered rocks are described in the same way as weathered rocks except that 'weathered' is replaced by 'altered' and in abbreviations, 'W' is replaced by 'A.'

In granitic and similar very strong crystalline rocks it is commonly possible and useful to subdivide the distinctly weathered materials further on the basis of dry strength of 50 mm diamond drill cores. The terms used are based on those of Moye (1955), defined as follows in Table 2.6.

Table 2.3. Descriptive terms for weathered condition of rock substance. Modified from McMahon, Douglas & Burgess (1975).

Term	Symbol	Definition
Residual soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported
Extremely weathered rock	XW	Rock is weathered to such an extent that it has soil properties, i.e. it either disintegrates or can be remoulded, in water
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron-staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock
Fresh rock	FR	Rock shows no sign of decomposition or staining

Table 2.4. Descriptive terms for strength of rock substance.

Rock strength class*	Symbol	Point load strength index $I_{s(50)}$	Approximate unconfined compressive strength Qu (MPa)
Extremely weak	EW	0.04	1
Very weak	VW	0.2	5
Weak	W	1	25
Medium strong	MS	2	50
Strong	S	4	100
Very strong	VS	10	250
Extremely strong	EH		

*Based on ISRM (1978): *Suggested methods for the quantitative description of discontinuities in rock masses.*

Table 2.5. Description of weathered or altered rock substance: Examples.

Rock type term	Rock condition term	Rock strength term	Abbreviated form
Granite	Fresh	Very strong	Granite FR (VS)
Granite	Extremely weathered	Soil properties (GW-GC)	Granite XW (GW-GC)
Granite	Distinctly weathered	Medium strong	Granite DW (MS)
Sandstone	Fresh	Weak	Sandstone FR (W)
Basalt	Distinctly altered	Medium strong	Basalt DA (MS)

Table 2.6. Weathered rock substance classifications for granite and similar rocks. Modified from Moye (1955), and Hosking (1990).

Term	Abbreviation	Description
Fresh	FR	Rock shows no evidence of chemical weathering
Slightly weathered	SW	Rock is slightly discoloured but rings when struck by a hammer; not noticeably weaker than fresh rock
Moderately weathered	MW	Rock is discoloured, produces only a dull thud when struck by a hammer; noticeably weaker than fresh or slightly weathered rock but dry samples about 50 mm across cannot be broken across the frabric by unaided hands
Highly weathered	HW	Rock is discoloured, can be broken and crumbled by hand, but does not readily disintegrate in water
Extremely weathered	XW	Material disintegrates when gently shaken in water, i.e. has soil properties

Lee & de Freitas (1989) propose a similar but more detailed classification scheme, for granitic rock substance in Korea.

2.5.2 *Limitations associated with any system for classifying weathered rock*

Quite often, particularly in sedimentary rocks, it is difficult or even impossible to determine which, if any of the rock substance at a site is fresh (unweathered). This is because such rocks can vary greatly in colour, porosity and strength due to past processes involved in their formation, rather than weathering processes.

Some rocks which are very weak or extremely weak when fresh (e.g. some shales and poorly cemented sandstones) assume soil properties (i.e. they classify as 'extremely weathered') when only slightly affected by chemical weathering. Intermediate weathered conditions cannot be defined in these cases.

Weathering products of carbonate rocks and rocks cemented by gypsum or anhydrite range from cavities or soils (due to solution) to very strong rock substances (due to re-deposition of material dissolved elsewhere in the mass). Such weathering products usually cannot be described adequately using the system on Table 2.3. Site-specific classifications are preferable in such cases.

2.5.3 *Classification systems for weathered rock masses*

The International Society for Rock Mechanics (1978) and British Standards Institution (1981) recommend the use of the descriptive terms and grades in Table 2.7, for the description of weathered rock masses, for engineering purposes. The ISRM recommends that the rock mass is described first in the broad terms given in that table, after which the weathered condition of rock substances can be described using the terms defined in Table 2.8.

Dearman (1986) proposes minor changes to the wording of Table 2.8 and to the application of the system, but suggests no changes to Table 2.7.

The authors consider that the above approach forms a basis for only a simplistic description of weathered profiles. It does not allow for the fact that many relatively common weathered rock masses comprise large proportions of rock which has been greatly weakened by weather-

Table 2.7. Weathered rock mass classification of ISRM (1978) and BSI (1981).

Term	Description	Grade
Fresh	No visible sign of rock material weathering: perhaps slight discolouration of major discontinuity surfaces	I
Slightly weathered	Discolouration indicates weathering of rock material and discontinuity surfaces. All material may be discoloured by weathering and may be somewhat weaker externally than in its fresh condition	II
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones	III
Highly weathered	More than half of the rock material is decomposed to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones	IV
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported	VI

Table 2.8. Weathered rock substance classification of ISRM (1978) and BSI (1981).

Term	Description
Fresh	No visible sign of weathering of rock material
Discoloured	The colour of the original fresh rock material is changed. The degree of change from the original colour should be indicated. If the colour change is confined to particular mineral constituents this should be mentioned
Decomposed	The rock is weathered to the condition of a soil in which the original material fabric is still intact, but some or all of the mineral grains are decomposed
Disintegrated	The rock is weathered to the condition of a soil in which the original fabric is still intact. The rock is friable, but the mineral grains are not decomposed

The stages of weathering described above may be subdivided using qualifying terms for example 'slightly discoloured', 'moderately discoloured', 'highly discoloured'.

ing, but do not necessarily contain any material which has been reduced to 'soil' properties. Hence the percentage of 'soil' material does not form a logical basis for classification of many exposures of weathered rock mass.

For example, a rock mass exposure of interbedded siltstone and sandstone which are very strong rocks when fresh, could comprise 80 percent siltstone which is distinctly weathered, very weak and 20 percent of sandstone which is distinctly weathered, extremely weak (using terms defined in Tables 2.3 and 2.4). This rock mass would be very much weaker than its 'fresh' equivalent, but using the system set out on Table 2.7 it would be described as 'slightly weathered' or 'Grade II.'

Similarly, a mass of fine grained granitic rock (extremely strong when fresh) comprising 50 percent moderately weathered and 50 percent highly weathered substance (Table 2.6) would be described as 'slightly weathered' or 'Grade II' using the definitions on Table 2.7. It is considered that these latter descriptions would be inappropriate.

Also, the ISRM (1978) advice to describe a rock mass in the broad terms of Table 2.7 before recording and describing its components as defined on Table 2.8 is considered illogical.

The authors conclude that (the ISRM 1978 and BSI 1981) rock mass classification systems do not meet the objectives defined in Section 2.5, and therefore do not recommend their use, except where broad generalizations are required. In these cases the grade numbers only should be used, to avoid confusion between the descriptive terms for mass and substance.

Lee & de Freitas (1989) describe a classification system for weathered granitic rock masses in Korea, which are usually 'transitional,' i.e. free of core stones. The classification incorporates their system for rock substance (See 2.5.1). Unfortunately they use the same terms and abbreviations, e.g. 'Moderately weathered' (MW) for both the substance (grade) and mass (zone). A zone in the mass is classified as moderately weathered if more than 50 percent of the substance in it is classified as moderately weathered. The system allows for very detailed description of 'transitionally weathered' granitic rock masses; probably more detail than is necessary in most embankment dam applications.

2.6 LANDSLIDING

At the site of any dam located across a river it is quite common to find evidence of past landsliding on at least one side of the valley. This is not surprising because every river valley will have developed by some combination of the following processes:

a) erosive downcutting by the stream, which unloads the materials underlying the valley floor and causes the valley sides to be steepened;

b) mechanical weathering processes (resulting from a) which weaken and usually increase the permeability of the materials under the valley floor and sides;

c) chemical weathering processes which further weaken the materials forming the valley sides, converting near-surface rocks partly or wholly to soils and generally lowering their mass permeability, and

d) soil creep, erosion, deposition, and rockfalls on the valley sides, giving rise to deposits of slopewash and scree, and to overall flattenning of the valley slopes.

The overall effect of a), b) and c) is to reduce the stability of the valley sides.

The valley-bottom profile on Figure 2.28 has developed by these processes, in granitic rocks and in a mediterranean or semi-arid climate. Because of the relatively dry conditions and a pattern of joints which favours stability, no landsliding has occurred or appears likely to occur.

However, if the joints were in less favourable orientations and the climate wetter, resulting in a higher water table, landsliding would be more likely to occur. Figure 2.29, taken from Patton & Hendron (1973), shows such a situation.

Wedge A B D, bounded by relict joints in the low permeability zone of residual soil and extremely weathered rock, may be subject to excessive water pressures from the underlying more permeable zone, causing it to slide.

A much larger, deeper slide would be possible if a continuous thin seam of extremely weathered granite parallel to surface AB was present within the more rocky Zone IIA, daylighting at the lower line of springs (S).

Deere & Patton (1971) describe weathering profiles developed in most common rock types and show how landsliding often occurs within them.

There are many different forms and mechanisms of landsliding and it is beyond the scope of this book to describe them. For useful accounts readers are referred to Varnes (1958, 1978), Selby (1982), Hunt (1984) and Hutchinson (1988).

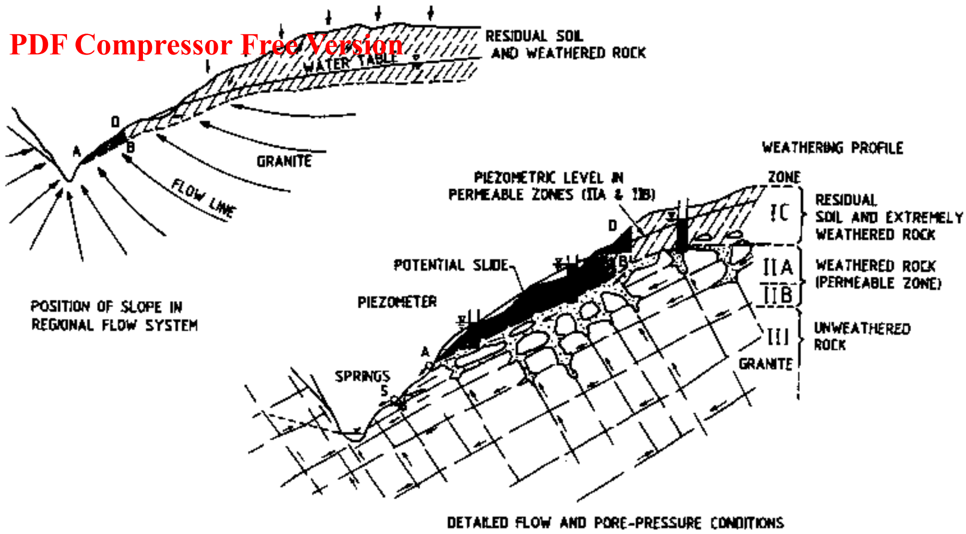


Figure 2.29. Typical conditions likely to result in landsliding of slope in weathered granite (based on Patton & Hendron 1973).

2.6.1 *'First-time' and 'reactivated' slides*

A distinction can be made between 'first-time' landslide activity which develops at 'intact' sites like that shown on Figure 2.29, and that which is simply reactivation of an old landslipped mass which has become stabilised or is subject to only minor creep type movements.

This distinction is important because there is a very large difference between the predictability of the two kinds of sliding.

2.6.1.1 *Reactivated slides*

Experience has shown that in more than 50 percent of cases, disturbance of an old landslide eg. by cutting into or inundating its toe, causes reactivation. Hence if there is clear evidence that all or part of a slope is an old slide mass, a high risk of reactivation by construction activities or operation must be assumed, at least until geotechnical conditions in the slope are sufficiently well known to prove otherwise.

2.6.1.2 *First-time slides*

First-time slides are much more difficult to predict. Reliable prediction is not possible without considerable knowledge of the geotechnical conditions in the slope, and of the nature of the proposed disturbing activities. For example, there is not enough subsurface data on Figure 2.29 for one to say definitely that Wedge A B D will slide, with or without some excavation above Point A.

2.6.2 *Importance of early recognition of evidence of past slope instability at a project site*

It is very important to recognise any evidence of past slope instability early in the planning of a dam project, for the following reasons:

a) Landslide debris comprising broken rock, soil or mixtures of soil and rock in an uncompacted state is usually not acceptable anywhere in the foundations of embankment dams. This is because of the potentially high, variable and often unpredictable compressibility and permeability of such materials. Also the deposit may contain, or be underlain by, slickensided failure zones or surfaces with unacceptably low shear strengths.

If these materials occur only in a relatively shallow deposit, it may be possible to remove them locally and expose a satisfactory foundation. However, as mentioned in Section 2.6.1, experience shows that in many cases where landslipped materials on slopes are disturbed by excavation, renewed landsliding occurs either in the materials themselves or from the source areas of the deposits.

Because of this, if all or a large part of one bank at a potential dam site is found to be underlain by such materials, it is usually best to adopt or look for an alternative site. If it is decided to persevere at the original site, a very thorough site investigation will be needed, the results of which may well cause the site to be abandoned.

b) If there is evidence of past landsliding very close to the proposed dam, and within the area of proposed associated works i.e. spillway, diversion works, quarry, borrow area and haul roads, then there will be a risk (usually high) of renewed slope movements during construction or more seriously, during operation of the project.

In such cases, it is best to adopt an alternative site without evidence of past instability, if one is available. Failing that, a very thorough site investigation will be necessary, the prime objective of which will be to assess the probability and likely effects on the works, of renewed instability.

c) If there are old stabilised landslides, or 'creeping' landslides in the proposed storage area, these could be reactivated during filling or operation of the storage. Such renewed landsliding may present the following types of hazard:

– partial or complete blockage of the storage, where this is narrow, to form a landslide dam. Flooding upstream, and overtopping and eventual failure of this landslide dam are further consequences. Failure of landslide dams can be disastrous (Schuster & Costa 1986, and King et al. 1987, 1989).

– overtopping of the dam by a large wave generated by a rapid slide, as happened at Vaiont Dam in Italy in 1963 (Hendron & Patton 1985, Muller 1964, 1968).

– damage to intake or outlet works or to roads located close to the storage.

– severe limitations on the operation of the project.

As for a) and b) above, comprehensive site studies would be necessary to determine the slide models and mechanisms and to assess the risks and hazards involved.

It should be clear that the presence of old landslides in any of situations a) to c) above raises questions which ideally should be answered during the feasibility stage and certainly by the end of the design stage of the project planning (see Chapter 4, Section 4.6).

Many old landslides are easily recognised during stereoscopic examination of aerial photographs, by their characteristic surface form and by anomalies in the local vegetation, soils or rocks.

2.6.3 *Dams and landslides: Some experiences*

This section describes some experiences with dams at sites showing evidence of past landsliding.

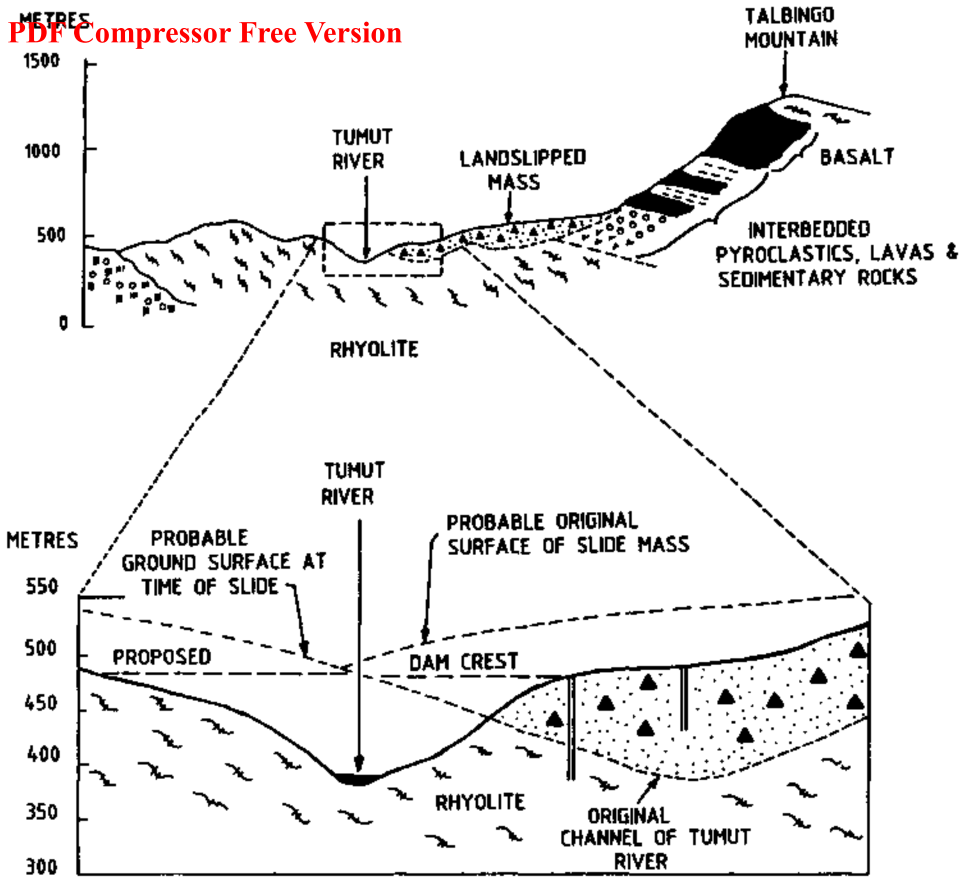


Figure 2.30. Original site for Talbingo Dam, cross section looking downstream.

2.6.3.1 *Talbingo Dam*

At the original site for Talbingo Dam, in the Snowy Mountains of New South Wales (Fig. 2.30) the right bank was formed almost entirely by the remnants of a mass of variably weathered basalt which had slipped more than a kilometre downslope. Initially, the landslipped basalt mass had dammed the Tumut River. Breaching of the landslide dam against the left side of the valley had apparently caused diversion of the river to its present course.

The geological picture on Figure 2.30 was deduced almost entirely from air photos and geological outcrop mapping on 1:2400 scale. It was confirmed by refraction seismic traverses and diamond drilling, which showed the slide mass to consist of extremely weathered basalt containing a small percentage of irregularly distributed blocks of less weathered basalt from gravel size up to several metres across.

An alternative site free of landslide evidence was adopted. At this site, more than 100 m upstream, the 161 m high Talbingo Dam (rockfill) was built, using 2.3 million m^3 of the basaltic landslide deposit for its impervious core. During operation of the borrow pit, some renewed sliding movements occurred within the deposit.

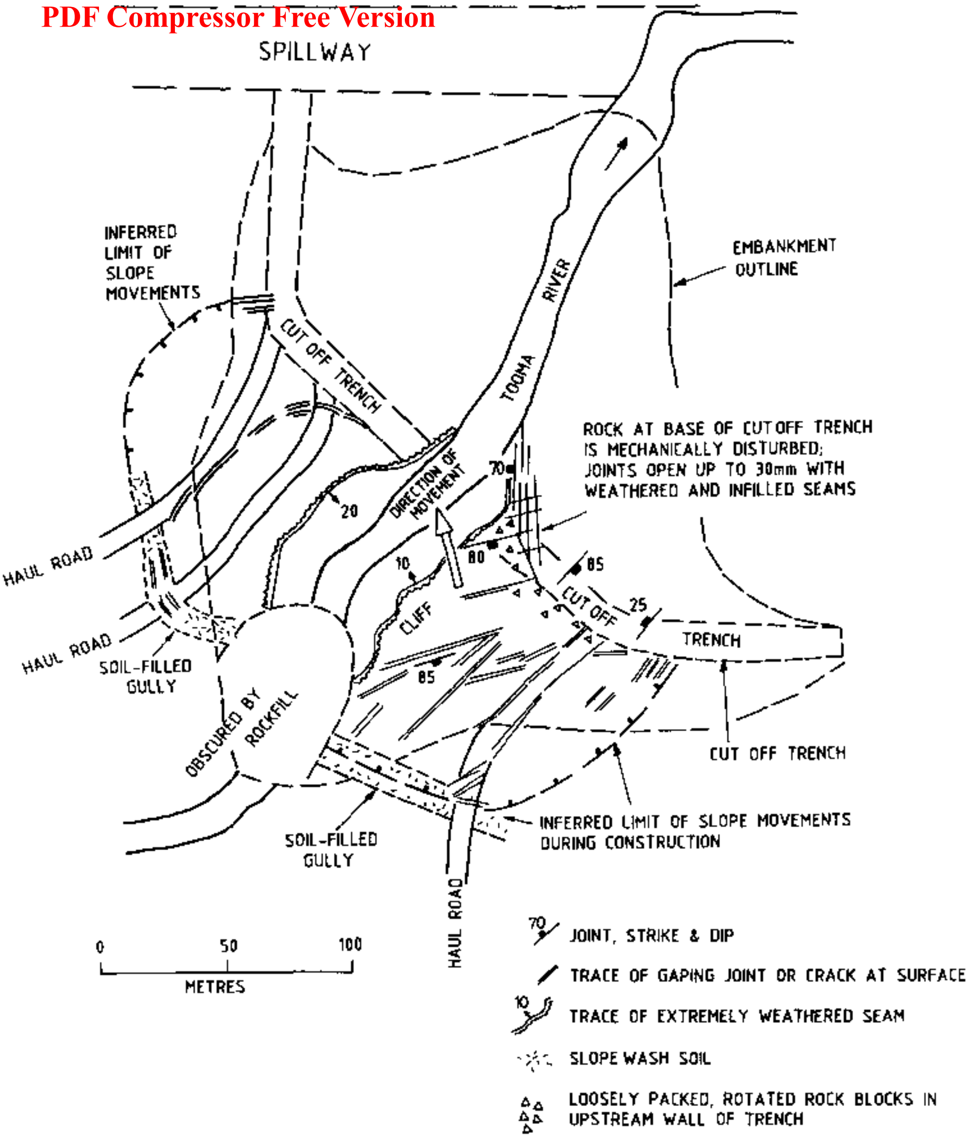


Figure 2.31. Tooma Dam, plan showing observed features and movements which occurred during construction.

2.6.3.2 *Tooma Dam*

Tooma Dam is a 68 m high earth and rockfill structure, constructed during 1958-1961 in the Snowy Mountains of New South Wales.

The dam is located in a steep-sided valley which has been entrenched about 80 m below an older, broad valley. Prior to construction, outcrops of granitic rocks were present along both banks of the river and extended locally to 10 to 40 m above river level. The largest outcrop area formed a cliff about 30 m high on the right bank beneath the upstream shoulder of the (proposed) dam (Figs 2.31 and 2.32).

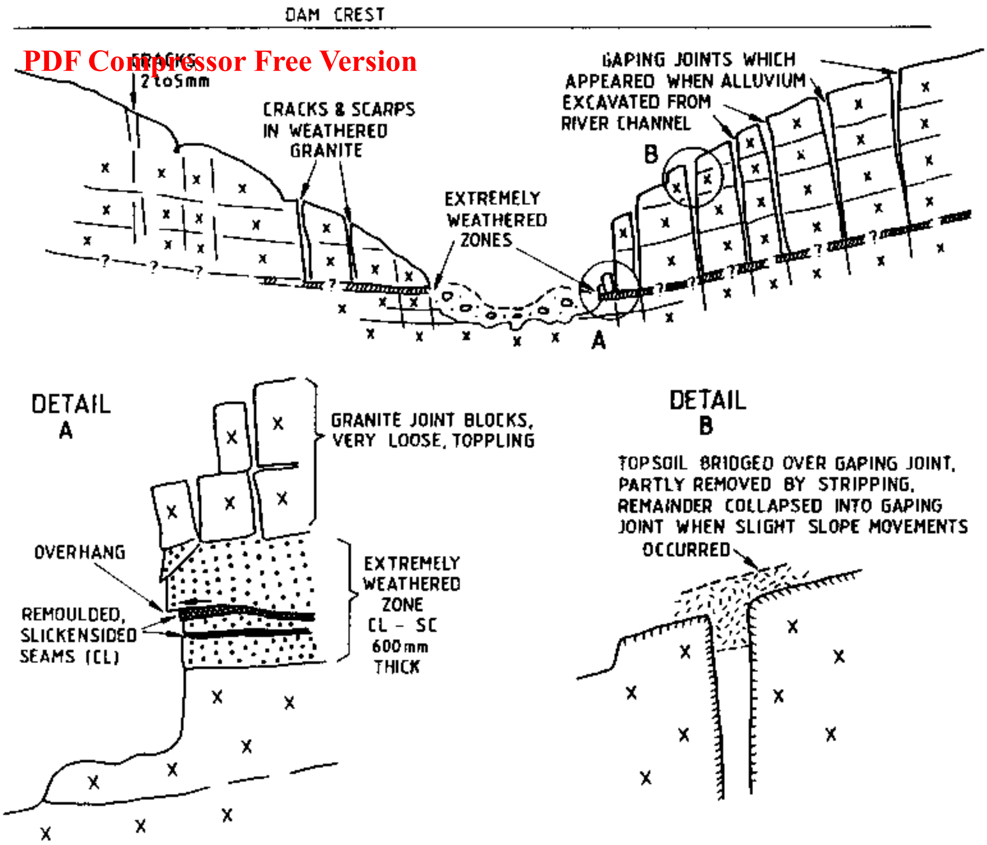


Figure 2.32. Tooma Dam, cross section and sketches through downstream shoulder.

This outcrop and a lower outcrop area directly opposite on the left bank, were terminated on their upstream sides by soil-filled gullies. The rock outcrops above river level showed near-vertical joints roughly parallel and normal to river direction.

During the planning stages subsurface exploration of the site included trenches up to 1.5 m deep cut by hand methods, and 17 diamond boreholes with water pressure testing. It was concluded from the results that the site would show wide variations in weathered profile, but no special problems were envisaged.

During stripping of the foundation, the variations in weathered profile were confirmed (Fig. 2.32). However the exposed rock was much more mechanically loosened than predicted. In the valley sides, the floor of the cutoff trench showed near-vertical joints parallel to the river which were open as much as 30 mm, apparently due to downslope block movements (in the past) along prominent joints dipping towards the river at 10 to 30 degrees. Clay-infilled joints and weathered seams next to joints were abundant.

The last specified stripping was removal of 2 to 3 m depth of boulders and gravel from the river channel. During this operation, many cracks open from 10 up to 500 mm appeared in and above the cliffs on the right bank.(Figs 2.31 and 2.32). The cracks were mapped to the upstream side of the cutoff trench where they became ill-defined in a steeply dipping zone of open jointed, clearly disturbed and weathered rock striking obliquely to the river (Fig. 2.31). Lesser cracks and scarps appeared in extremely weathered granite on the left bank.

At the same time, 400 to 700 mm wide zones of extremely to highly weathered granite which were wet and showed local seepages became exposed close to river bed level on both banks. Both zones were undulating but dipping generally between 10 and 20 degrees towards the river. Each zone contained one or more remoulded seams which included surfaces with slickensides (Fig. 2.32A). In the right bank seams these were trending downslope and obliquely downstream. Displacements of 50 mm were observed across one slickensided seam during a period of two days. Monitoring ceased when several cubic metres of rock toppled from immediately above the weathered zone at the monitoring site.

When all of these features appeared the contractor withdrew from the upstream part of the site until it could be demonstrated that large-scale landsliding was not imminent.

Close examination of the largest cracks on the right bank showed that these had formed by the collapse of surface soils into pre-existing gaping joints which appeared to extend almost down to the extremely weathered zone (Fig. 2.32). Thus, although the cumulative past downslope extension appeared to be between 1 and 2 m, the amount of current movement appeared to be much smaller, probably just enough to cause the bridging soils to crack and fall into the already gaping joints. This was confirmed by precise survey which was started after the appearance of the cracks, and showed only about 150 mm of further movement over a period of about 2 weeks.

It was concluded that the gently dipping joints were probably sheet joints and that the basal weathered zones had been formed by weathering along the lowest of these joints. The pre-construction block movements towards the river were probably initiated by stress relief, and continued due to water thrusts and gravity.

On each bank the largest of the old movements appeared to have occurred between the upstream soil-filled gullies and the cutoff trench (Fig. 2.31).

Movements of the right bank cliff area were stopped by early placement of rockfill in the river bed. Some of this is visible at the right hand edge of Figure 2.33.

Changes to the dam design were made during construction, to allow for the potential for water to penetrate along open joints under the upstream part of the the earth core, to minimise the possibility of adverse deformation of the core, and to allow the unstable slope to be quickly restrained. The adopted measures included

– steepening of the upstream face of the earth core from 1:1 to O.25H:1V (Fig. 2.34);

– addition of a secondary downstream grout curtain, and blanket grouting between the two curtains (Fig. 2.34);

– extending the lower half of the cutoff trench about 25 m downstream, and immediate commencement of placing the upstream rockfill, in advance of the earth core zone placement.

In addition to the above, extensive dental treatment of open joints (with mortar) was carried out in the cutoff trench, and backfill concrete was placed over the badly disturbed and weathered rock at the upstream edge of the cutoff trench on the right bank, as shown on Figure 2.33.

Further details of the above are provided in Hunter(1982) and Hunter & Hartwig (1962).

Pinkerton & McConnell (1964) describe the performance of the embankment during construction and its first 2 years of operation and show that it behaved well.

Although it was possible to modify the design during construction and a good result was achieved, there must have been some cost to the project because the evidence of past slope movements was not recognised and allowed for during the early planning. It is considered that adequate evidence was there, but was not found or recognised, mainly because the question of

Figure 2.33. Right bank at Tooma Dam after completion of foundation stripping.

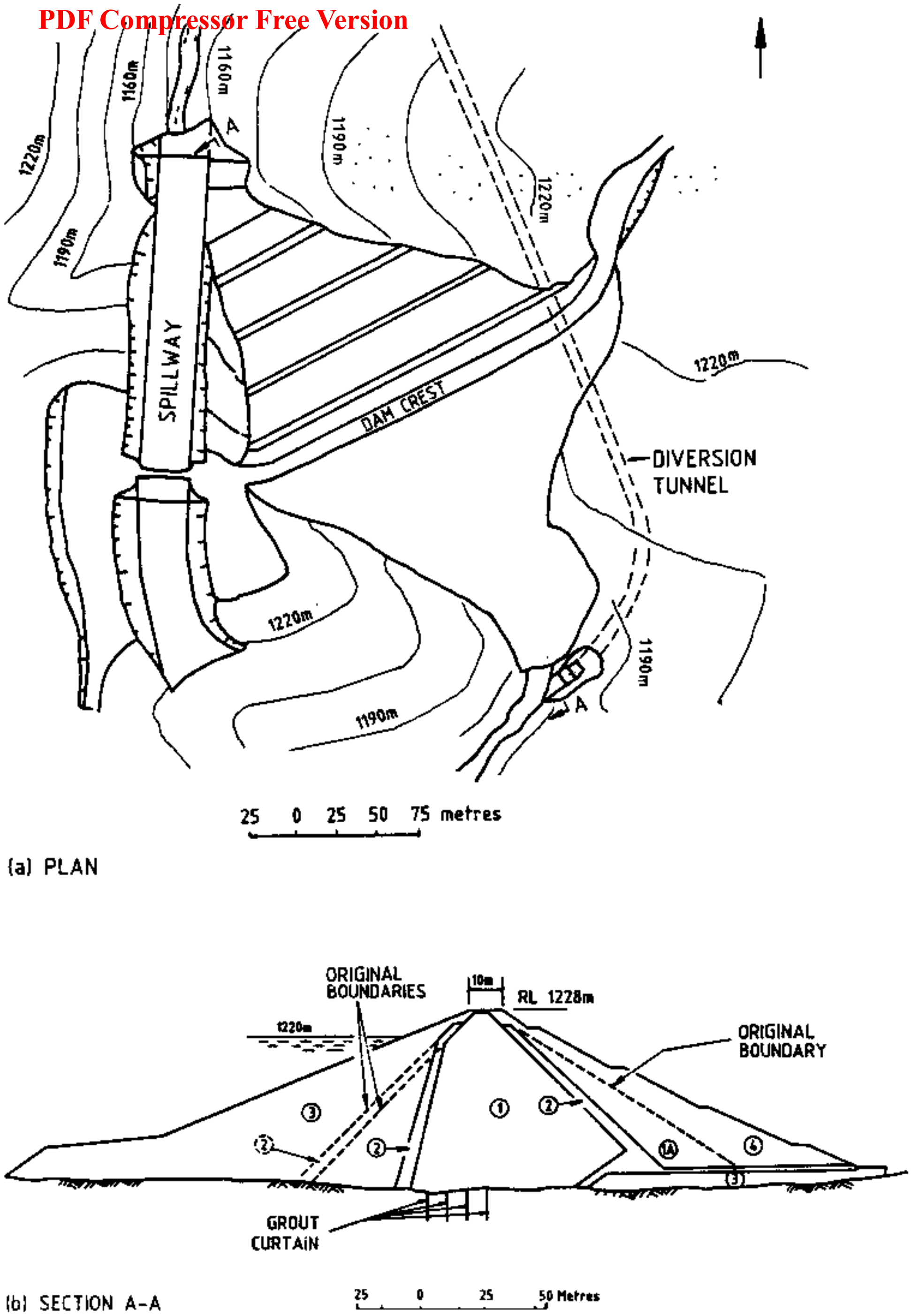


Figure 2.34 . Tooma Dam, cross sections of original and rezoned embankment (from Hunter 1982).

possible valley-side movements was not addressed. In later projects deep trenches excavated by bulldozer and ripper have usually been successful in locating the gaping or infilled joints indicative of this kind of slope instability.

2.6.3.3 *Wungong Dam*

Wungong Dam is a 66 m high earth and rockfill structure built during 1976-1979 near Perth, Western Australia. The site is underlain by granite containing several intrusive dykes of dolerite. The rocks are variably weathered. Lilly (1986) describes landslides which occurred during construction.

During early site investigations, a rounded scarp was recognised on the right bank above the upstream shoulder of the proposed dam, and noted as a 'suspected old landslide' (Fig. 2.35). Trenches below this feature confirmed that the ground was disturbed, but the areal extent and depth of the disturbed mass was not determined.

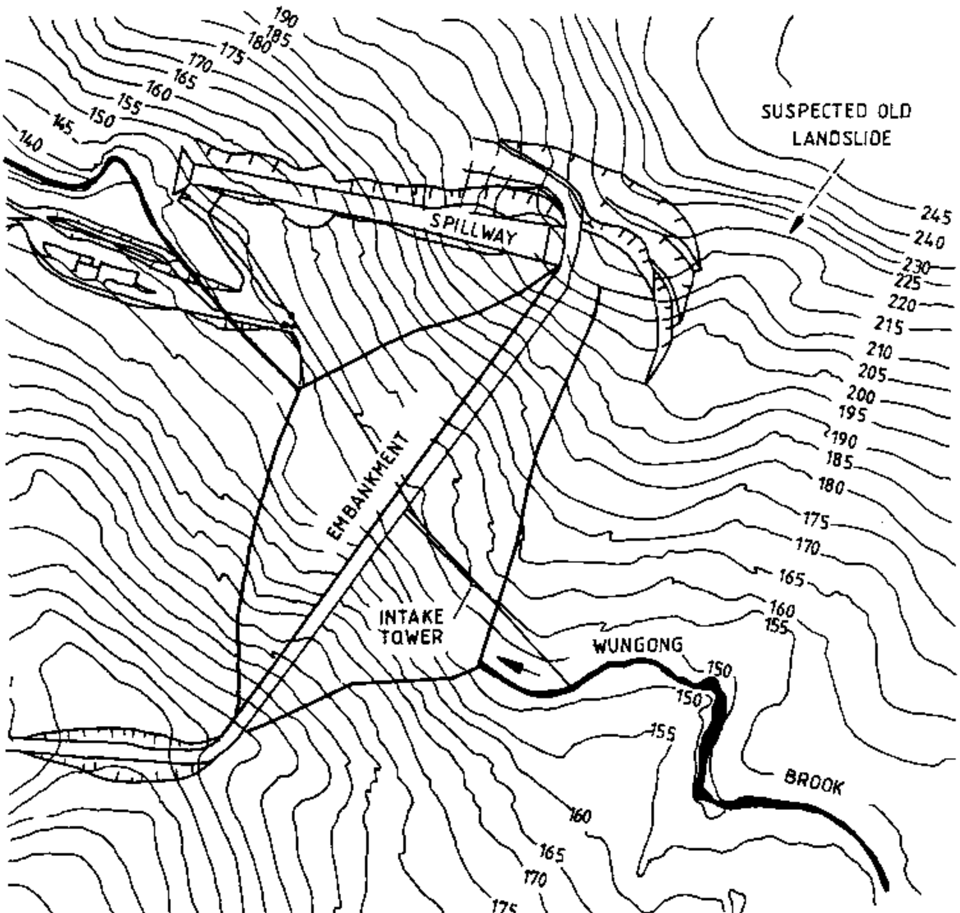


Figure 2.35. Wungong Dam, plan showing adopted layout and inferred approximate position of old landslide (from Lilly 1986).

To allow for this suspected landslide the dam axis was rotated to bring the upper right shoulder downstream of it (Fig. 2.35). It was planned to investigate the area further during construction, and it was expected that much of the landslipped material near the embankment would be removed during excavation of the spillway approach channel.

During stripping of the foundation and placement of fill, four landslides occurred from the area below the suspected old slide. The slides cut through a construction access road, and the

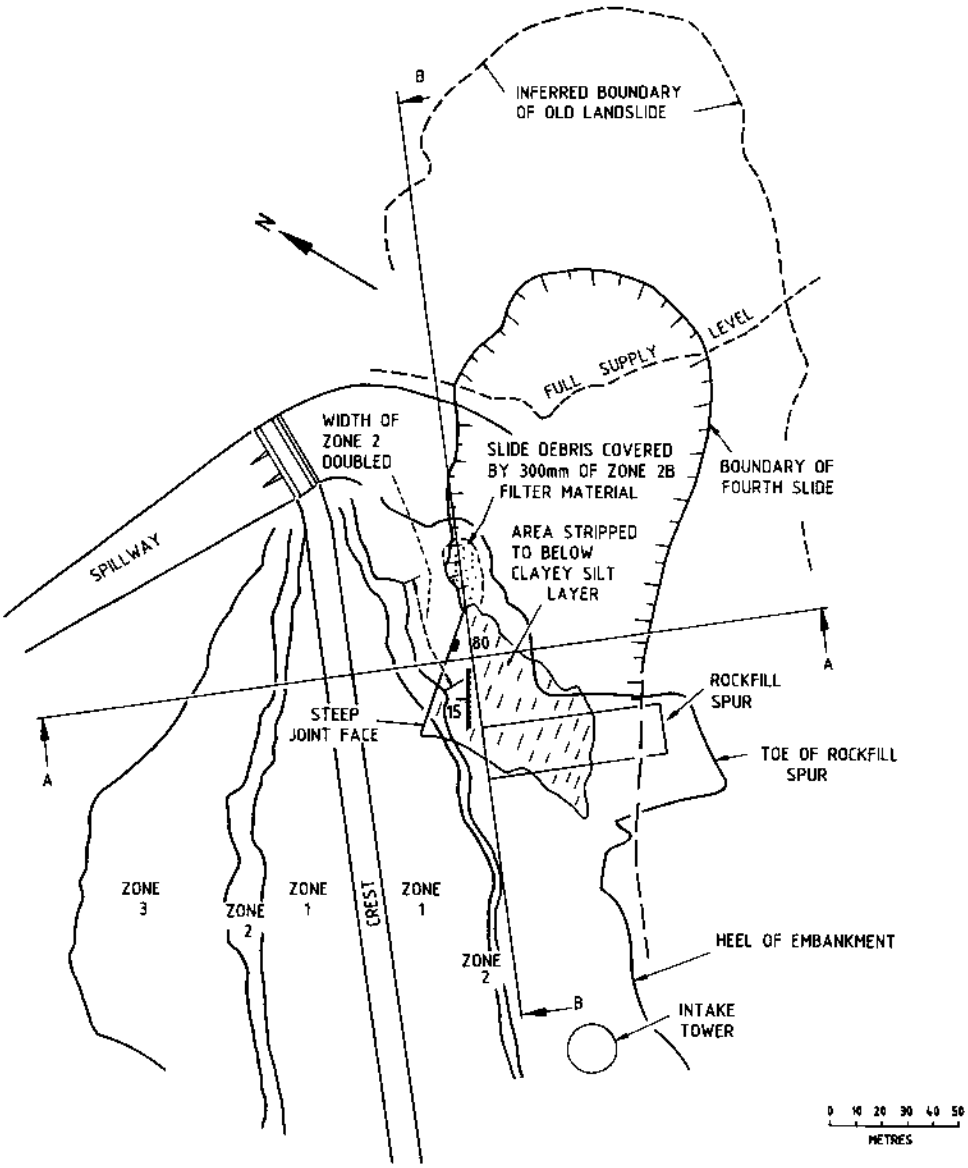


Figure 2.36. Wungong Dam, right abutment, plan showing the extent of the fourth landslide and remedial works. (based on Lilly 1986).

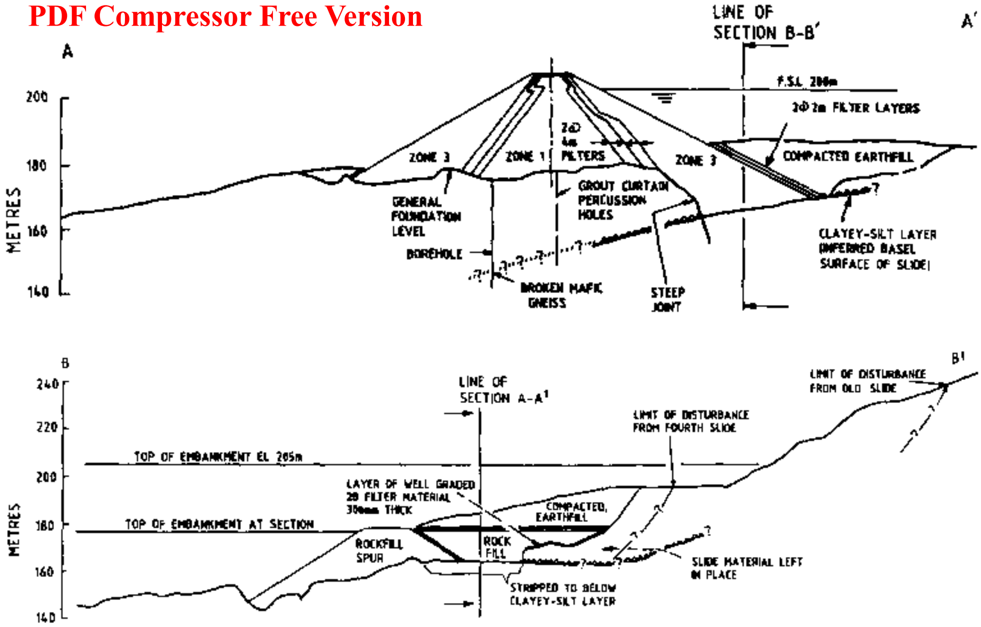


Figure 2.37. Wungong Dam, sections showing remedial works (based on Lilly 1986).

fourth and largest slide (about 100 000m^3) cut through the spillway approach channel and moved into the downstream rockfill and filter zones (Lilly 1986).

At this stage it was appreciated that the stability of the dam embankment was also in question, and a major investigation involving 17 diamond drillholes was carried out.

Excavation of the slide material in the foundation area showed that the basal failure surface here was a seam less than 50 mm thick of micaceous clayey silt, probably a weathered and sheared, basic dyke. The seam was slightly undulating and dipping almost directly downstream (i.e. beneath the dam) at about 15 degrees. The slide mass here was a wedge bounded by this seam and a joint striking roughly at right angles to the river, dipping 80 degrees upstream (Fig. 2.36). Movement had been directly towards the river at the wedge intersection plunge angle of about 5 degrees (Fig. 2.37, Section BB). Although water pressures obviously contributed to the sliding, an extremely low shear strength was indicated for the seam.

After removal of some rockfill which had already been placed, a 40 by 70 m area of the rockfill foundation was excavated down to below the seam level (Figs 2.36 and 2.37). The slide was then stabilised by a spur of rockfill which later formed a projecting part of the dam shoulder. Other modifications to the dam included a local widening of the downstream filter zones and placement of a filter blanket over an area of slide material left in place beneath the rockfill (Figs 2.36 and 2.37).

Monitoring during filling showed local downslope movements of up to 90 mm in and upstream from the spillway approach area. Lilly (1986) explains that these were to be expected, as cracks up to 150 mm previously noted in these areas would be closing up. No significant unusual movements of the dam embankment were recorded.

Although a satisfactory dam was the end result, the landsliding and discovery of disturbed materials extending into the rockfill and filter foundations caused serious questions to be raised, and delays to construction. With hindsight, more thorough investigation and delineation of the slide during the planning stages might have resulted in the dam being located well away from the affected area.

2.6.3.4 *Sugarloaf Dam*

Sugarloaf Dam is an 85 m high, 1000 m long concrete faced rockfill structure, constructed near Melbourne, Australia during 1976-1979. It impounds an offstream storage of 95 000 megalitres.

The site geology and its influence on the design and construction of the dam are discussed in Stapledon & Casinader (1977), Casinader & Stapledon (1979) and Casinader (1982).

The site is in a region of broadly folded siltstones and sandstones of Devonian age which are distinctly weathered to depths of more than 30 m below ridges and plateaus.

Figure 2.38 shows the layout of the embankment which extends across the valley of Sugarloaf Creek and then for 500 m along the crest of a ridge. No other site was acceptable topographically.

During the feasibility stage, detailed geological mapping of surface features and trenches dug by bulldozer and ripper were the main investigation activities.

After initial excavation the floors of the trenches were cleaned carefully by small backhoe and hand tools to ensure that all minor defects such as open and clay-filled joints and thin crushed seams were clearly exposed.

This work showed that much of the foundation area on both banks was formed by dipslopes; thinly interbedded and weathered siltstone and sandstone were dipping parallel to the ground surface (Fig. 2.38). Dip and slope angles ranged from 12 to 30 degrees. On the right bank a

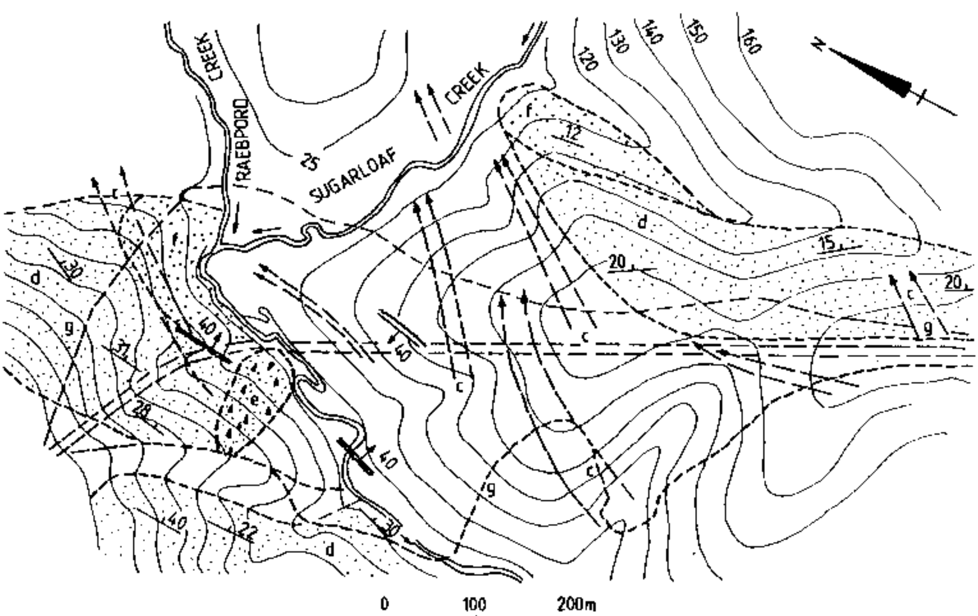


Figure 2.38. Sugarloaf Dam, geological plan showing dipslopes. c) axes of minor plunging folds; d) dipslope, slightly disrupted; e) scar of old landslide; f) dipslope, very disrupted; g) outline of embankment.

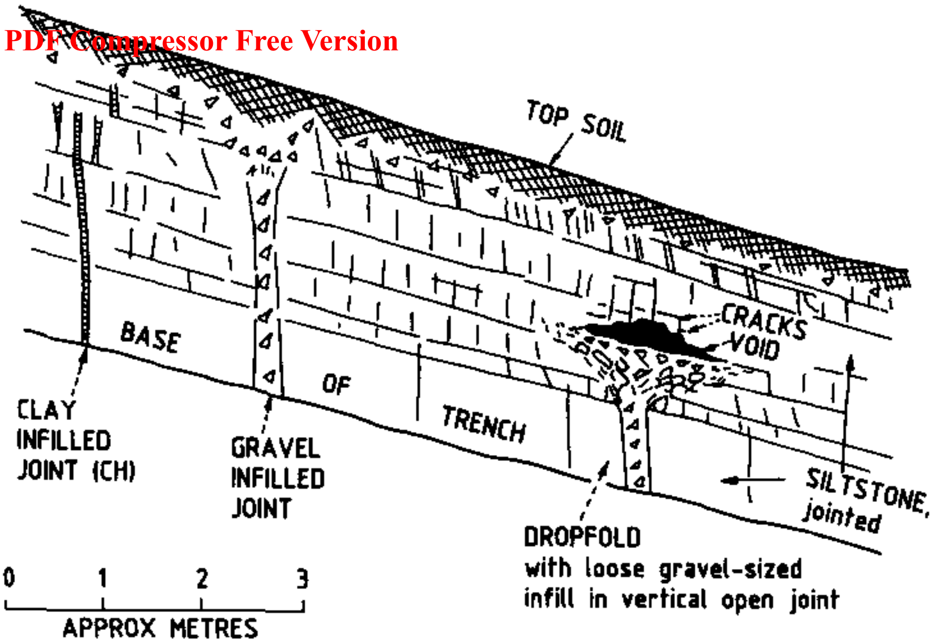


Figure 2.39. Features exposed in trenches on a left bank dipslope at Sugarloaf Dam.

suspected old landslide was confirmed by the trenching to be underlain by deep, disturbed soil. With the exception of this feature the valley slopes in the foundation area showed no topographic evidence of past landsliding or of significant undercutting of the dipslopes.

Trenches cut straight down the dipslopes showed the bedding surfaces to be very smooth and that slickensided seams of low plasticity clay 2 to 20 mm thick were present along them at spacings of 1 to 2 m. The seams were highly dispersive. Diamond core drilling showed that the clay seams were present to depths of more than 10 m and that they were the weathered equivalents of bedding surface faults (crushed seams, see Chapter 3, Fig. 3.19a and Section 3.5.3) which were present at about the same spacings, at greater depths.

Also present in some parts of the trenches were near-vertical features of the kind shown on Figure 2.39. These were mainly joints which were open or infilled with gravel or up to 20 mm of high plasticity clay. Also present near the base of a left bank dipslope was a 'dropfold' structure where an upper bed had clearly cavitated into a gaping joint in a lower bed. Figure 2.40 shows this feature.

It was judged that these near-vertical features had been formed by extension of the near-surface beds during downslope sliding movements along weathered bedding surface seams. Cross-cutting defects near the base of the slope and in some places past undercutting by erosion had apparently provided freedom for the slope movements to occur.

Laboratory direct shear tests showed the weathered bedding surface seams to have effective residual strengths of about 10 degrees. The residual value was adopted in stability analyses because the seams were initially near-planar faults and the small displacements during slope movements would have been enough to reduce their strength to this value.

The combination of very low strength dispersive seams and dipslopes provided interesting

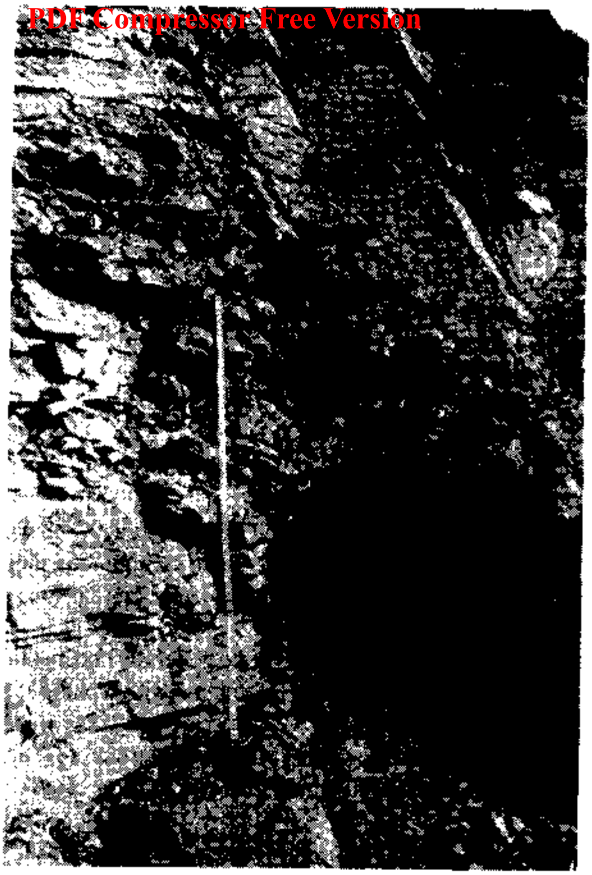


Figure 2.40. Dropfold structure exposed in trench on the left bank at Sugarloaf Dam.

design questions which are discussed in Casinader & Stapledon (1979). Figure 2.41 shows the arrangement adopted for the section of the dam on the left bank ridge. The plinth was located close to the crest of the ridge, to minimise loading of the dipslope. This resulted in much of the embankment being located on the downstream side of the ridge. The dipslope was stabilised by covering it, and filling the gully upstream from it, with waste material from a nearby excavation.

During construction the conditions found were essentially as predicted. Minor landsliding occurred on the right bank dipslope when the surface soil and vegetation was stripped, and again when a 4 m deep trench was cut directly down the slope. These occurrences under very low normal stress conditions confirmed the need to use residual strength in the design.

Stability problems on the right bank proved more difficult than predicted, but early recognition of this during construction enabled the design here to be modified with minor disruption of the works.

The evidence of thin seams and past slope movements at Sugarloaf was quite subtle and may not have been found if it had not been consciously looked for and exposed in well-prepared, deep trenches.

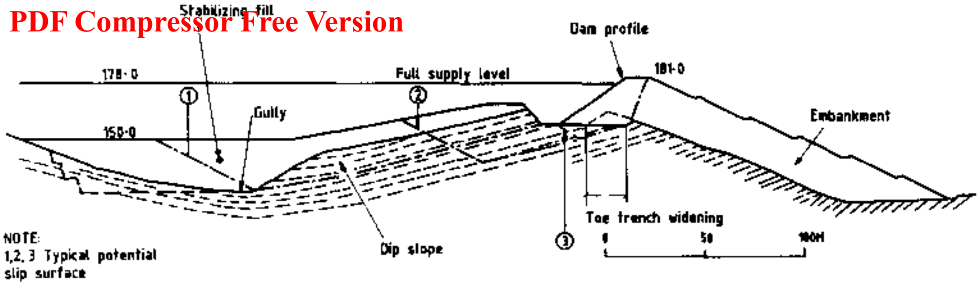


Figure 2.41. Sugarloaf Dam, cross section through left abutment ridge section.

2.6.3.5 *Thomson Dam*

The 166 m high Thomson Dam is an earth and rockfill structure located 120 km east of Melbourne, Australia. It was constructed between 1977 and 1984.

The site is underlain by a folded, interbedded sequence of siltstone and sandstone, similar to that at Sugarloaf. However the folds are tighter and more complicated than at Sugarloaf, there are many normal and thrust faults, and bedding-surface sheared and crushed seams are spaced generally at less than 1 m intervals.

Figure 2.42 shows the major folds and faults at the site. Fold axes trend generally north-south and plunge northwards. A major feature at the site is the Thomson Syncline which plunges to the north at about 12 degrees.

Although this fold pattern was recognised during the feasibility and pre-construction design stages, its potential to contribute to instability of slopes and foundations was not appreciated at that time.

As at Sugarloaf, the site was trenched extensively during the feasibility investigations but the trenches were not so deep, and the same standard of cleanup was not achieved. However the Ski-jump slide (Fig. 2.42) on the upstream left abutment was identified from highly disturbed rock in a trench cut into an area showing typical landslide topography.

Diamond drilling in the upper right bank core foundation area indicated closely jointed, seamy and partly weathered rock down to 70 m. High losses occurred during water testing of this zone and it was suspected that it may have been disrupted by past landsliding. This was allowed for in the designed depth (up to 30 m) of the cutoff trench on the upper right bank. Also, the owner, Melbourne and Metropolitan Board of Works, decided to excavate the whole of the cutoff trench on this bank, prior to awarding a contract for construction of the dam. During this excavation, and excavation of associated haul roads, it was found that past landsliding in the site area had been more widespread than expected and had caused disruption of the rock mass in the three areas shown on Figure 2.42. In each area the sliding had occurred where the combined effects of valley erosion and the fold shapes had caused downslope dipping beds to daylight. Details of each slide and the nature of the disturbed materials are given in Marshall (1985).

The Ski-jump Slide proved to be a relatively small feature. The folded rock here daylighted into a gully immediately upstream (Fig. 2.42). The slide mass was buttressed effectively by placing rockfill in the gully and the rockfill shoulder of the dam was built over it.

The Core Trench Slide proved deeper than expected, downslope movements having occurred along folded beds daylighting at river level. The trench excavation ranged from 10 to more than 40 m deep and involved more than a million cubic metres of excavation, all by

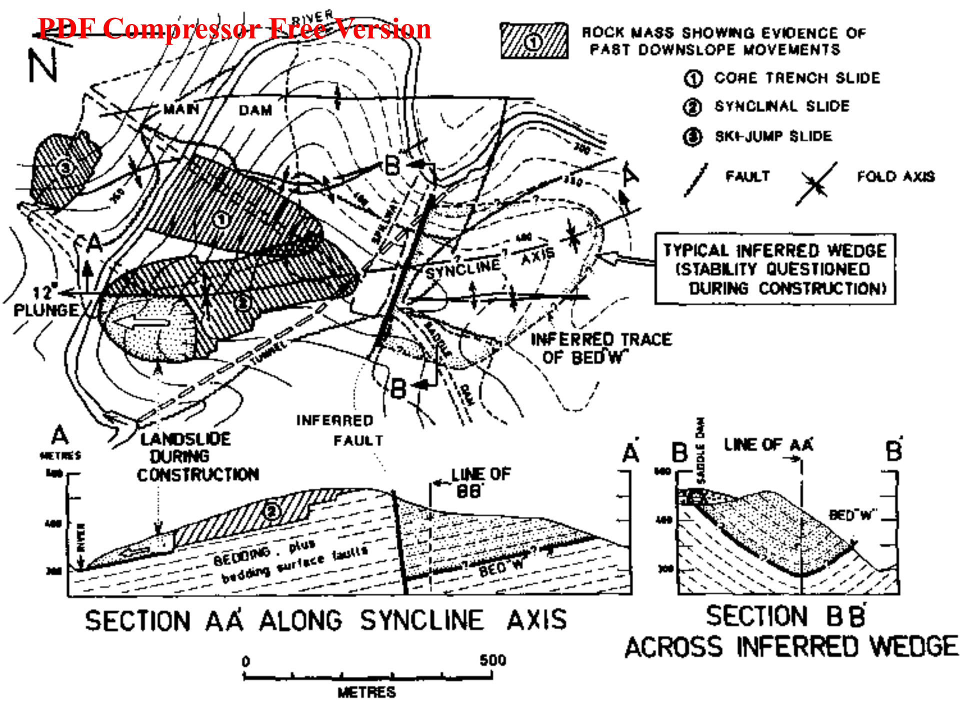


Figure 2.42. Thomson Dam site, geological plan and sections showing the main structural features.

Caterpillar D9 bulldozer without ripping. In the upper half of the trench deeper excavation appeared likely to give rise to instability of the sides and so it was decided to leave the deeper, moderately disturbed rock in place and treat it with a 5-row grout curtain. This rock contained many joints which were open or clay-filled, from 1 to 20 mm. It was appreciated that most of the clay would remain after the grouting but could be washed out in the long term, when the reservoir was filled. To allow for this possibility a reinforced concrete gallery was built in a slot cut into the upslope half of the cutoff trench. The gallery provides access for monitoring and for drilling and grouting equipment if regrouting should ever be needed.

The Synclinal Slide (Fig. 2.42, Section AA) was discovered during construction when cracks and displacements appeared in haul roads near the toe of the ridge at the right bank upstream shoulder of the dam. Movements in the toe area ranged from 1 mm/day in dry weather to 3 mm/day after rain. Movements stopped after 9 months on completion of a 50 m high stabilising rockfill built against the ridge toe. This required diversion of the river into a channel cut into the left bank, as the diversion tunnel was still under construction.

Detailed investigations showed that past movements in the ridge had extended right up to its crest. The sliding had occurred directly down the 12 degree plunging axis of the Thomson Syncline. Upslope from the toe area the base of the disturbed rock zone was stepped as shown diagrammatically on Figure 2.42, Section AA. In the ridge crest area rock disturbed by this slide occurred in the top few metres of the gate shaft and also at the top of the cutoff trench.

Changes to the project resulting from this slide included construction of a permanent stabilising fill of more than 2.6 million m^3, inclusion of special construction joints in the gate shaft, and elimination of an access bridge from the gate shaft area to the upper outlet tower.

During detailed studies of the Core Trench Slide and Synclinal Slide the succession of rock types and the shape of the Thomson Syncline became well known in those areas. Projection of this known geological picture southwards raised serious questions about the stability of a ridge on which the spillway and part of the saddle dam were located (Fig. 2.42, Section BB). Using residual strengths obtained from back analysis of the Synclinal Slide, this ridge appeared to have safety factors of less than unity against sliding along any of the numerous bedding surface seams which daylighted above river level. A major new site investigation was carried out over a period of 16 months, including 1.6 km of trenches, adits 250 and 460 m long, diamond drilling and field and laboratory testing. This work is described in detail by Marshall (1985), who showed that irregularities in the fold shape and rotational displacements on faults effectively precluded failure of the ridge. The following additional works and changes were undertaken, to further guarantee its stability:

– Weathered, potentially unstable rock was removed from the top of the ridge

– The axis and grout curtain of the saddle dam were moved 10 m upstream, to reduce the embankment load on the active wedge.

– Additional drainage galleries were driven into the ridge and connected by a drainage curtain of vertical boreholes.

The site for Thomson Dam is geologically very complex and was difficult to explore due to its steepness, dense vegetation and often deep soil cover. Some evidence of the slope stability problems at the site was found during the early planning and used in the design of the dam. The owner's decision to excavate the right bank cutoff trench prior to awarding the main construction contract (Hunter 1982) was a good one. As Hunter (1982) points out, this decision was more or less in accord with the approach of Terzaghi & Leps (1958) at Vermilion Dam. Terzaghi and Leps concluded that the foundations at Vermilion were so complex that further more detailed exploration would still leave a wide margin of interpretation. They therefore advocated proceeding with construction of the dam as designed on the best interpretations and assumptions, with careful monitoring of the construction, and modification of the design to suit the conditions as found.

However, at Thomson, after construction was well advanced the expenditure of money, technical manpower and time on geotechnical site investigations far exceeded that spent in the feasibility and design stages. Most of this effort went into investigation of the downstream ridge, the stability of which was a question that should have been answered in the feasibility stage.

If the construction stage studies had found this ridge to be unsafe, the options for remedial works were very limited, extremely expensive and would themselves have required further detailed site investigations to prove their feasibility. As discussed in Chapter 4, Section 4.6, the authors consider that for all dams, large and small, it is vital that sufficient funds are made available at the feasibility stage to ensure that all questions affecting feasibility are asked and answered satisfactorilly. For dams of the size of Thomson the exploration will often include adits as well as extensive and carefully cleaned trenches.

CHAPTER 3

Geotechnical questions associated with various geological environments

As explained 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 present chapter discusses eleven 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 generalizations 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 generalizations may not be valid at that particular site.

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 (Fig. 3.1). Many of these joints are often 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.

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

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.

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 subbase 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 blanketting 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 2.20 to 2.23. 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 (Fig. 2.23). 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, failure occurs wholly or partly along relic joints or other defects (see Chapter 2, Section 2.6 and Fig. 2.29).

The landsliding at Tooma and Wungong dams (Sections 2.6.3.2 and 2.6.3.3) occurred along local 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 – Checklist*

– 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:

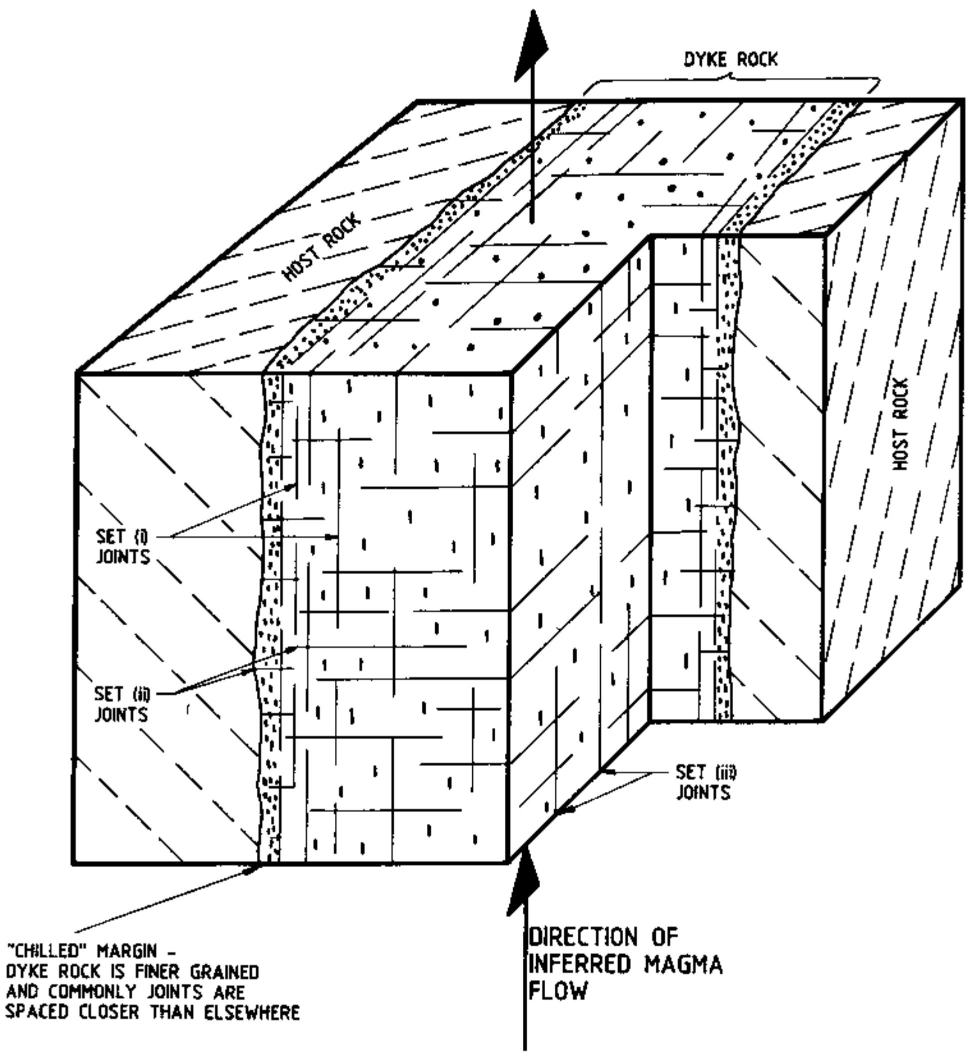


Figure 3.2. Some features commonly seen in dyke of intrusive rock.

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.

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 ie. 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.

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 & Poldervaart (1967), Francis (1976) and Bell (1983). 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 to 1000 m depths in Hawaii show little or no sign of compaction and are highly permeable. As the flow (Fig. 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.

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;

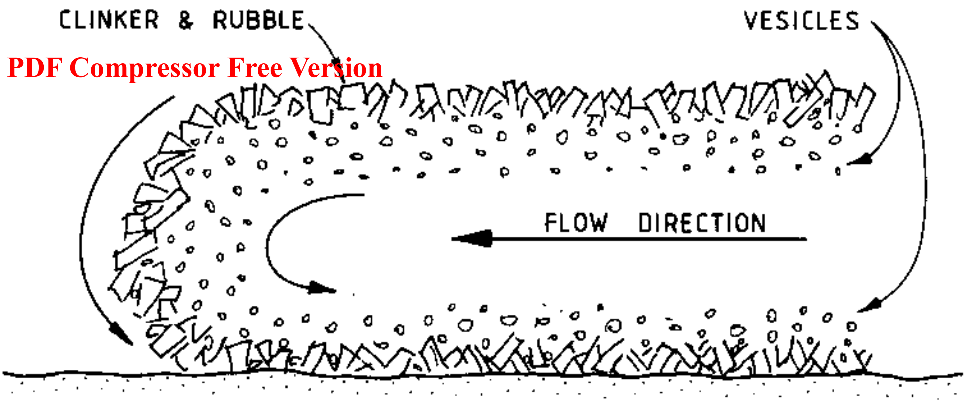


Figure 3.3. Diagrammatic longitudinal section through flowing lava showing the 'caterpillar track' mechanism which results in upper and lower layers of vesicular lava and in some cases clinker or breccia.

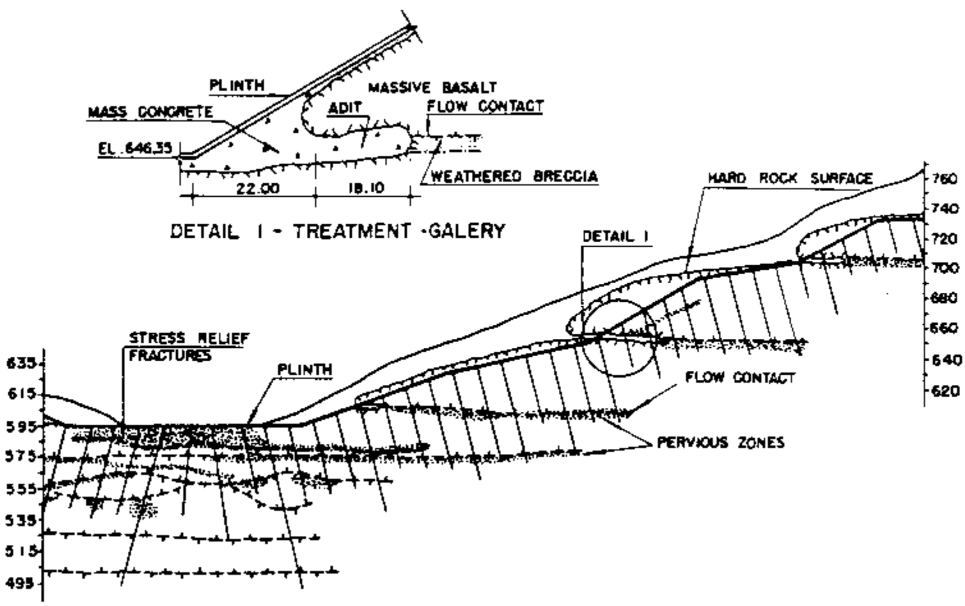


Figure 3.4. Geological section along plinth (right bank) at Foz Do Areia Dam. From Pinto et al. (1985).

– 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 (Fig. 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.

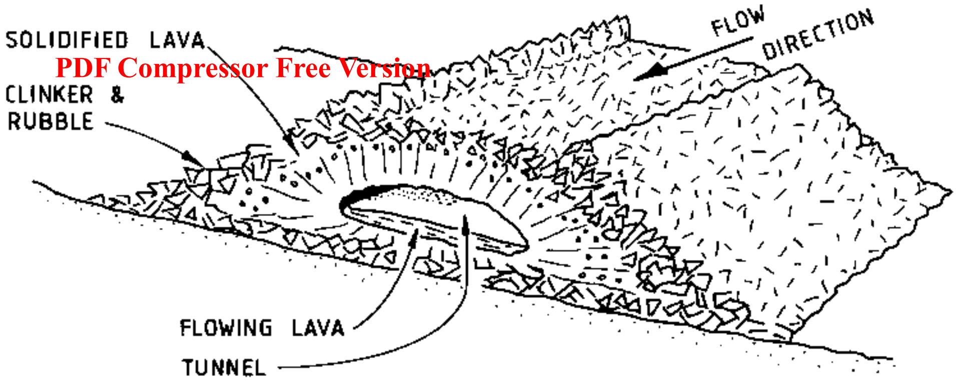


Figure 3.5. Perspective view of, and cross section through a lava flow, showing a lava tunnel developed when the lava flows forward faster than the supply.

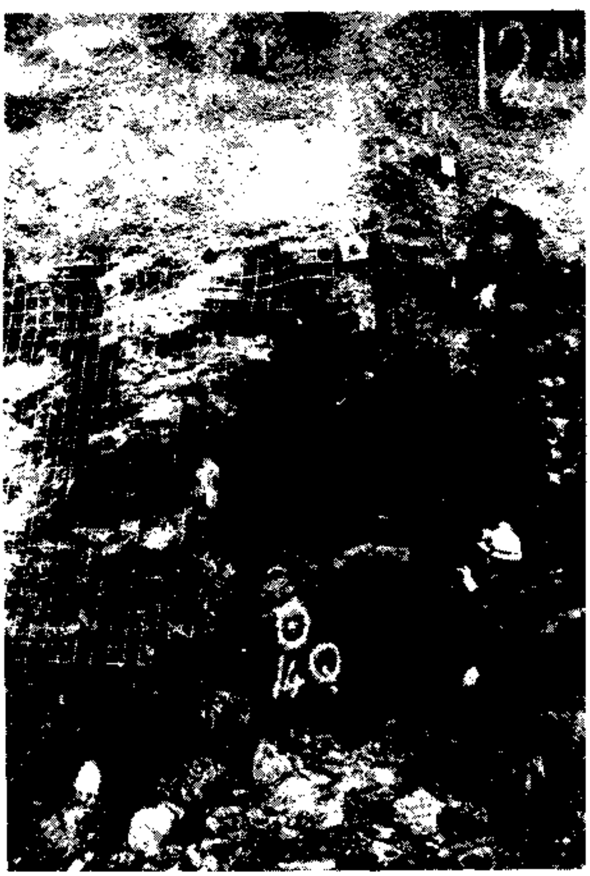


Figure 3.6. Lava tunnel in basalt, Hoppers Crossing, Victoria, Australia.

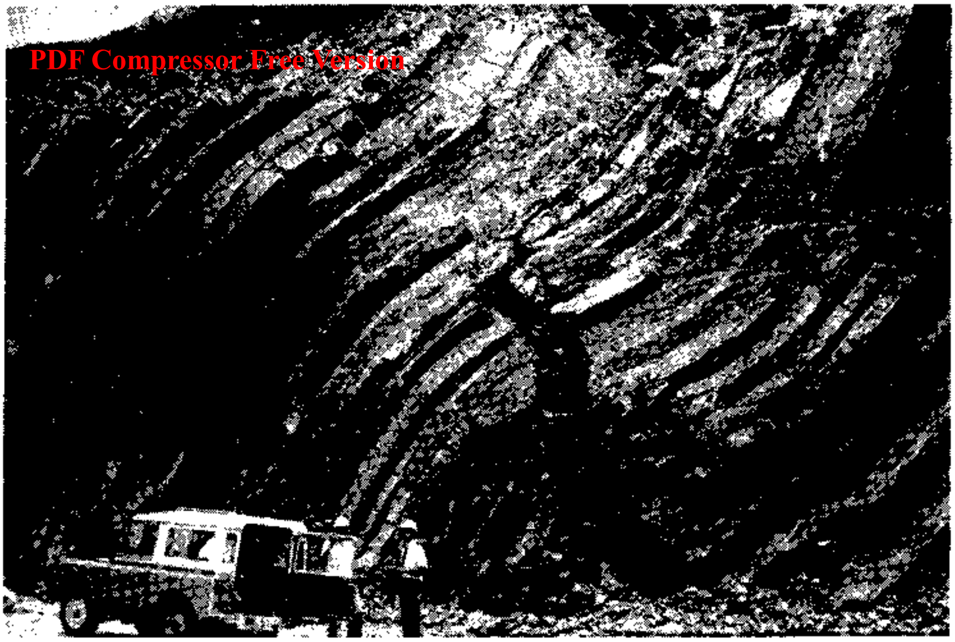


Figure 3.7. Columnar joint pattern in rhyolite, and younger intrusive dyke. High Island Dam, Hong Kong.

Thick lava flows sometimes contain 'lava tunnels,' which are circular, ovoid or lenticular in cross section, up to 15 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' (Fig. 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, eg. due to mechanical weathering close to steep valley sides, the rock mass permeability can be high.

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 m) 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

Figure 3.8. Pillow structure in basalt, Pembrokeshire, UK.

low modulus (see Table 16.4), 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 (Fig. 3.8).

The pillows may be welded together, separated by fine cracks, or separated by sedimentary and/or lava-derived detritus. The long axes of the pillows are generally roughly parallel to the boundaries of the 'flow.' Bell (1983) 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, then 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 & Van Atta (1986), Van Atta & Ludowise (1976), Cole & Beresford (1976), Cole & Sandy (1980), and Hosking & Tubey (1969).

As the alteration is caused by hot waters 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 (see Chapter 6, Section 6.1.3). 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 groundwaters exiting from a permeable zone beneath the extremely weathered

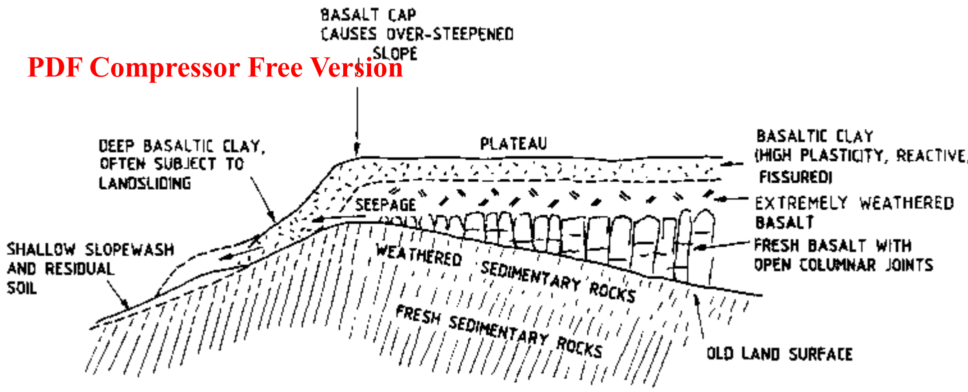


Figure 3.9. Typical profile through margin of basalt plateau, showing conditions which lead to slope instability.

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.

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 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.

Gillott (1975, 1986) and McConnell et al. (1950) describe observed effects of concrete expansion due to alkali aggregate reaction, and discuss the mechanisms involved. Stark & De Puy (1987) describe observations and tests on affected concrete at five dams in USA, and Cole & 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 & 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) – Checklist 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?
– Akali-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 'nuees 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

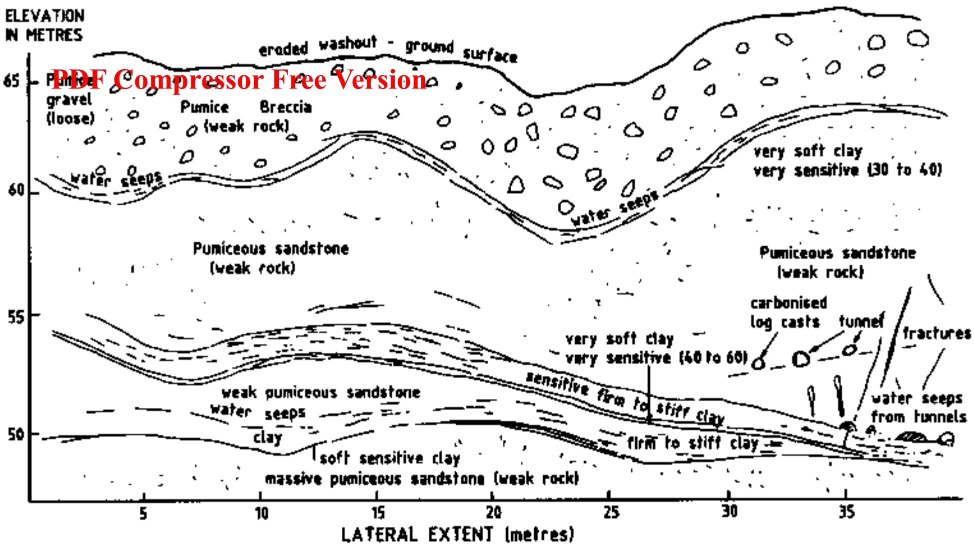


Figure 3.10. Exposure of pyroclastic materials in the collapsed area of Ruahihi Canal, New Zealand. From Prebble (1983).

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 indicate 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 mudflow;

– hydrothermal alteration, and chemical weathering.

The alteration and weathering products are clay minerals which may include montmorillonite, noted for its low shear strength, 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 Fig. 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 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 cause the strength contrasts in the pyroclastic substances to be greatly reduced.

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 degrees. The soils are mainly allophane and halloysite clays, with very high residual friction angles (most between 25 and 35 degrees).

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

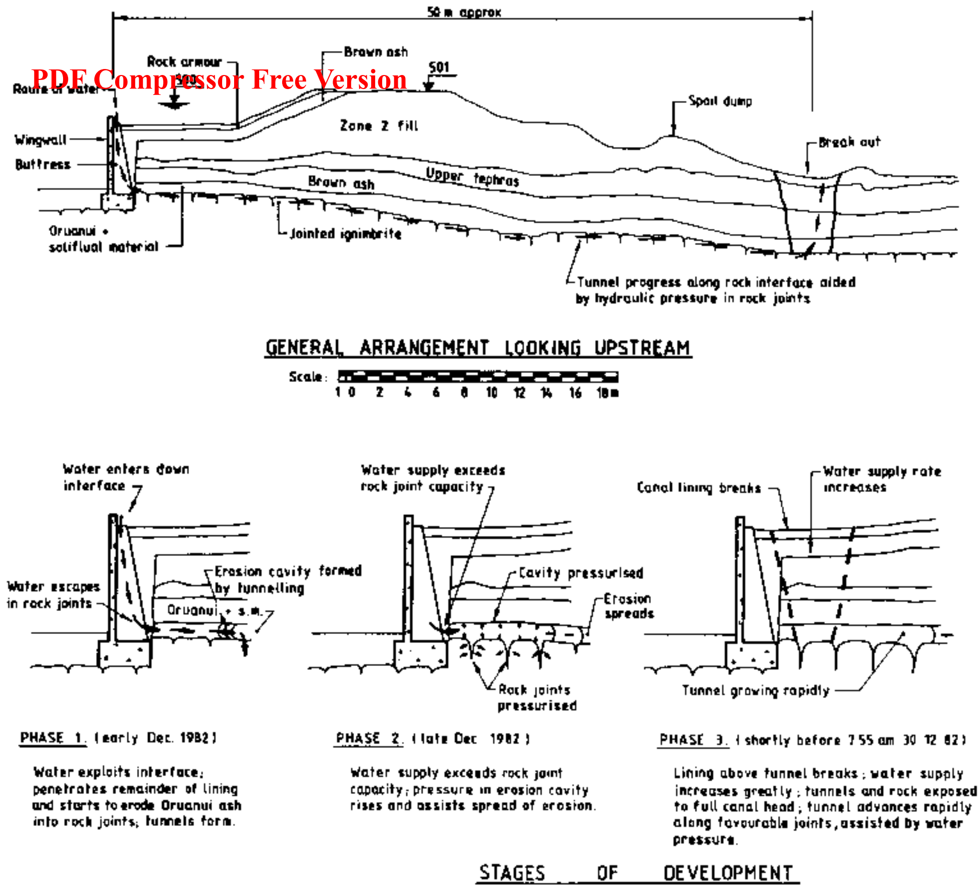


Figure 3.11. Wheao Canal failure. Diagrams showing the general arrangement and how the failures developed (from Jones 1988).

during compaction. However based on the experience at the 60 m high Matahina rock fill 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: it appeared to behave like a very low strength rock. This strength developed due to 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 & Newton (1988), included the possible reinforcement of the transition zones by 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 bulky shape and in the range 300 to 600 mm (see 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 (See Section 3.2.6).

3.3.3 *Pyroclastic materials – Checklist of questions*

– Extreme variability?
– Very low *in situ* densities: Collapse type behaviour?
– High *in situ* permeability?
– Brittle insitu and when compacted?
– Highly erodible insitu 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 schist 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, and 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*

3.4.1.1 *Strength and anisotropy*

Most schistose rocks have (minimum) 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,

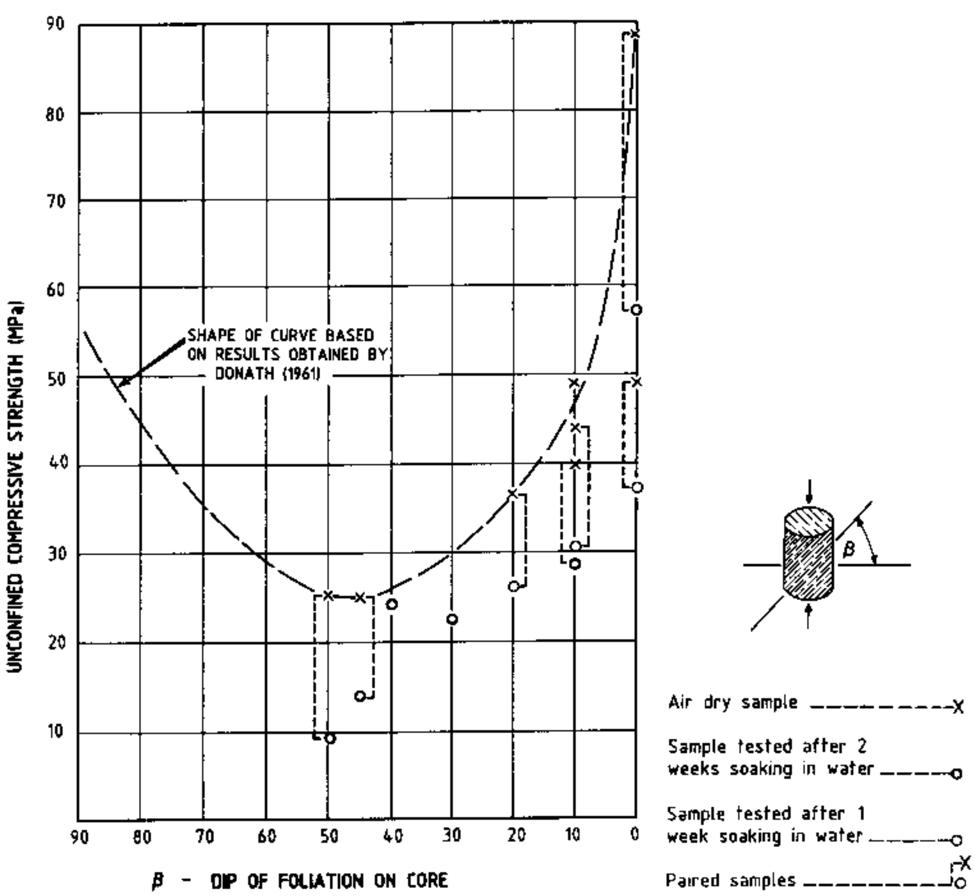


Figure 3.12. Variation of unconfined compressive strength of schist with angle between the foliation planes and applied loads (from Trudinger 1973).

South Australia. This schist is stronger than most; it contains generally about 40 percent of quartz and felspar, and about 60 percent sericite and chlorite. It is foliated but not exceptionally fissile. It can be seen that the strengths recorded for samples loaded at about 45 degrees 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, and 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 (see Chapter 6, Section 6.3).

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 to 65 percent reduction in strength, after soaking in water for 1 to 2 weeks. This result is typical of schistose rocks.

3.4.1.2 *Durability when exposed*

Many of the mica-rich schistose rocks are not highly durable when exposed to the weather – they tend to deteriorate at exposed surfaces due to either slaking, decomposition of sulphide minerals, or electrostatic moisture absorption (see Chapter 2, Sections 2.4.1, 2.4.4 and 2.4.5).

3.4.2 *Weathered products and profiles developed in schistose rock*

Schistose rocks vary widely in their susceptibility to chemical weathering. Varieties rich in

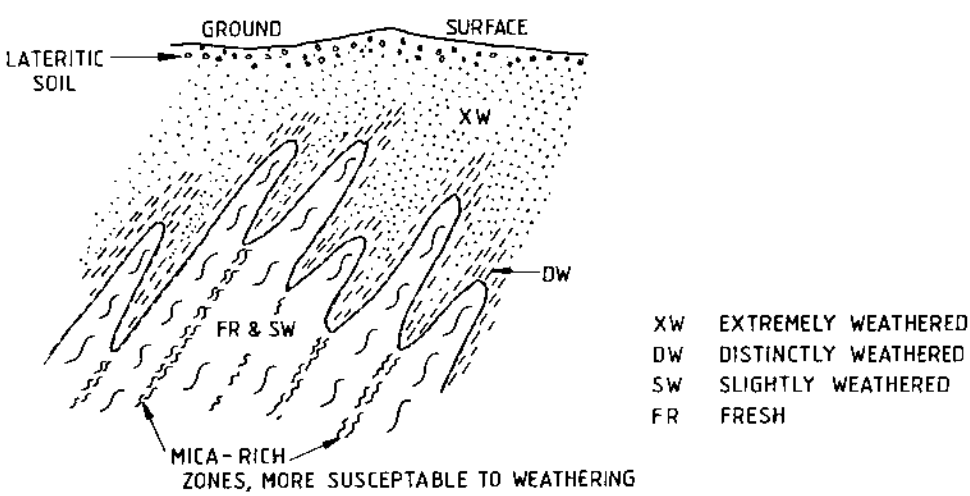


Figure 3.13. Weathered profile developed on schistose rock with steeply dipping foliation.

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. As a result of this, 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 insitu 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.

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 flakey shapes of the crushed materials, and inadequate strengths of the particles. Kammer & Carlson (1941) describe the unsuccessful use of phyllite as aggregate for concrete in a hydroelectric 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 to 0.9 m thick layers and compacted dry with a Caterpillar D8 tractor and by trafficking by the 50-tonne dump trucks. Post-construction creep settlements 5.5 years after completion ranged from 0.3 to 0.7 percent 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 construction procedures, the behaviour of the rock, and the fill densities obtained, in some detail. 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 (Fig. 3.14). Because the schist suffered a large strength loss on saturation (see Fig. 3.12) the rock was heavily watered during placement. During compaction by 4 passes of a 10-tonne vibrating roller, the uppermost 50 to 300 mm of most layers were crushed into a gravelly sandy silt (Fig. 3.15). These layers formed thin permeability barriers but they were readily penetrated by the basal rocks of the next layer and hence they did not form weak or compressible zones in the dam. Four years after completion, the maximum creep settlement was 110 mm or 0.2 percent of the embankment height.

Wherever schistose rocks have been strong enough to use as rockfill, and have been exca-

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 compaction by 4 passes of a 10-tonne vibrating roller (from Trudinger 1973).

vated after many years in service, they have been found to show no significant deterioration, at depths of greater than 2 m in the fill. See also Chapter 2, Sections 2.4.4 and 2.4.5.

3.4.5 *Structural defects of particular significance in schistose rocks*

There are several defect types of particular significance in schists, 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 degrees (Fig. 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 (ie. excluding the effects of large scale roughness), to lie in the range 7 to 15 degrees. Such defects commonly form the initiating failure surfaces of landslides in schistose rocks (see Fig. 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, eg. 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

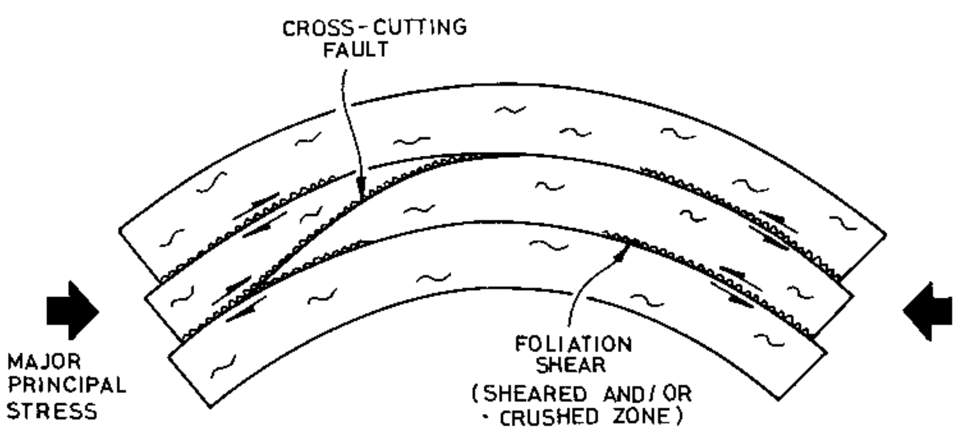


Figure 3.16. Probable way in which foliation shears and cross-cutting faults are formed, in some schistose rocks.

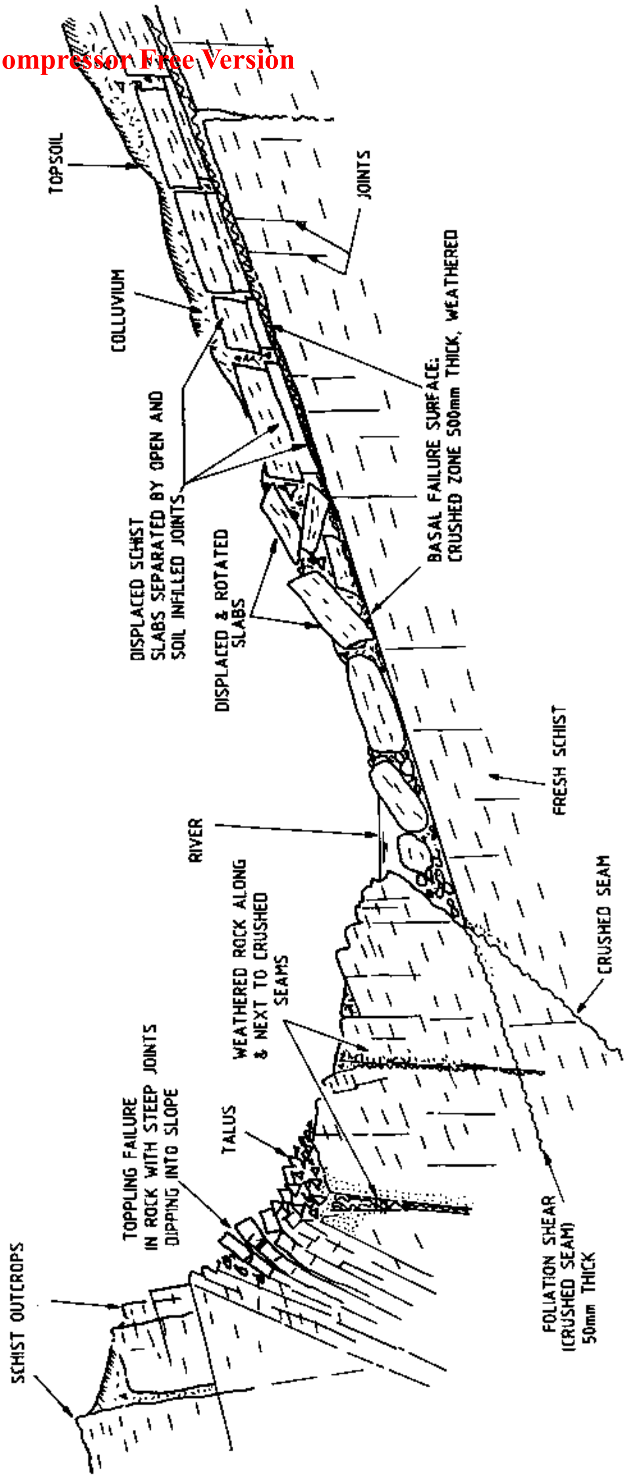


Figure 3.17. Typical valley profile in gently dipping schist affected by past landsliding along a foliation shear.

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 Dam, in New Zealand.

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 (Fig. 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 (Fig. 3.18b). Such kink bands can be considered as a special class of minor fault.

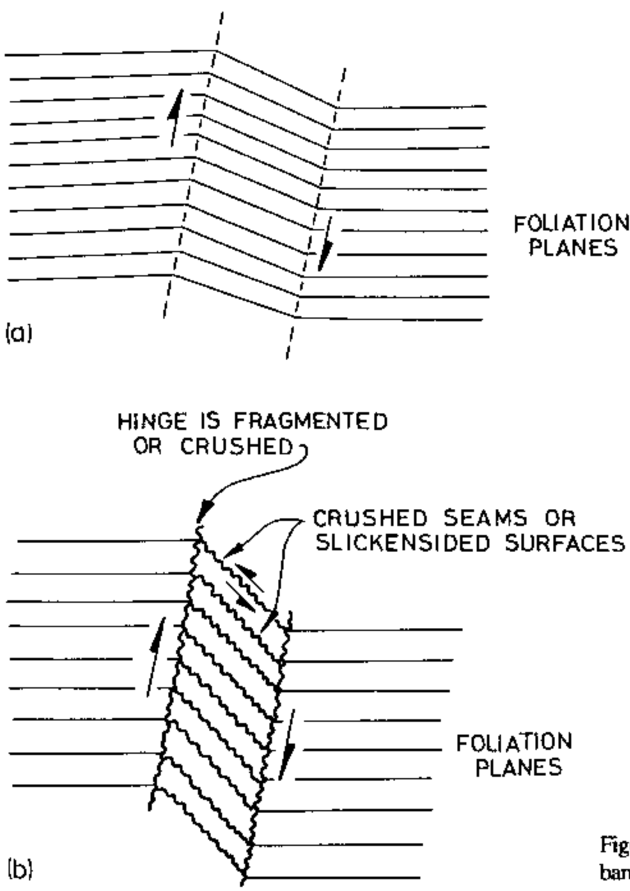


Figure 3.18. Formation and nature of kink bands.

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, USA (Hadley 1978), the Downie Slide in British Columbia (Piteau et al. 1978) and the Tablachaca Slide in Peru (Arnao et al. 1984, and Deere & 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 & Hancox (1992) and Riddolls et al. (1992).

3.4.7 *Schistose rocks – Checklist 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?
- Dusty working conditions?

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 as follows:

– 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).

3.5.1 *Engineering properties of mudrocks*

Most mudrocks when fresh lie in the weak to extremely weak range, as defined on Table 2.4. The extremely weak claystones grade into hard, overconsolidated clays, and in many cases would be more precisely described as such. 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 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, which cannot strictly be described as rock, also swell noticeably on removal of overburden from them.

The mechanisms involved in deterioration of mudrocks on exposure have been described already in Sections 2.1.2 and 2.4.1.

Because of their instability when exposed, special care needs to be taken during preparation of embankment or plinth 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.

Review papers of Taylor & Spears (1981) and Cripps & Taylor (1981) provide useful information about the engineering properties of mudrocks occurring in the United Kingdom.

3.5.2 *Possible presence of evaporites in some mudrocks*

Some reddish-brown rock sequences comprising mainly mudrocks (known as red-beds) have probably been deposited in inland sedimentary basins or sheltered coastal lagoons under arid or semi-arid conditions. Such rocks commonly contain evaporite minerals such as gypsum, anhydrite and halite (common salt). These minerals occur either within the rock substances as cements, as layers or beds within the sequence, or as veins filling once-open cavities, cracks, or joints. Stapledon et al. (1963) record such occurrences in mudrocks and sandstones of Triassic-Jurassic age near Vientiane (Laos). As these evaporite minerals are highly soluble in fresh water it is important that their presence in dam foundation or reservoir rim areas should not go undetected during site investigations. Chemical analyses of drilling fluid returns and of rock and soil samples are advisable in such areas.

If evaporite minerals are proven in a dam foundation their significance needs to be assessed carefully. The mechanisms of solution of evaporite minerals are discussed by James (1977), James & Lupton (1978) and James & Kirkpatrick (1980). See also Section 3.7.4.

James & Kirkpatrick (1980) describe the likely behaviour in dam foundations, of rocks and soils containing evaporite minerals. They also present guidelines on site investigation methods and suggestions on the seepage control measures required for dams built on such materials.

3.5.3 *Bedding-surface faults*

Valley bulging (see Chapter 2, Section 2.1.3 and Fig. 2.8) 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

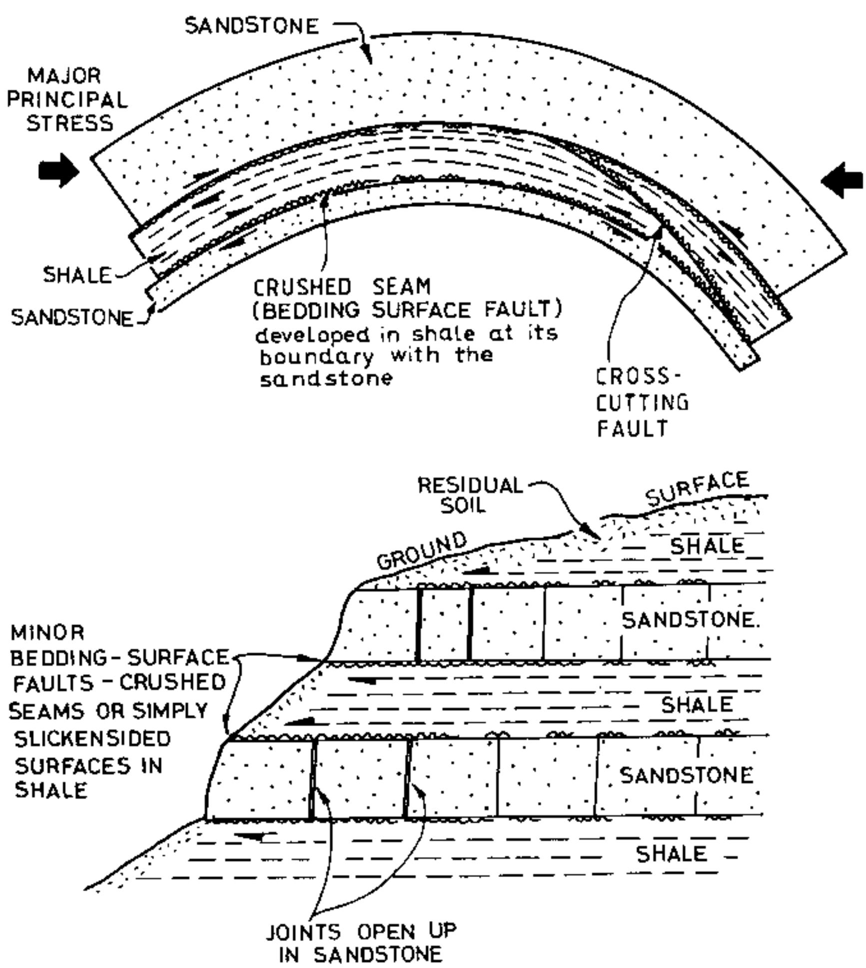


Figure 3.19. Usual ways in which bedding surface faults are formed in mudrocks

interbed slip during the movements, in the same way as foliation shears develop in schistose rocks (see Figs 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 degrees, 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 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 3.5.7).

The influence of bedding surface faults on the design of several embankment dams is described by Casinader (1982).

3.5.4 *Slickensided joints or fissures*

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

- Syneresis (Skempton & Northey 1952, White 1961).
- Shrink and swell movements (Corte & 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, orientations and continuity of the joints, need to be taken into account.

3.5.5 *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. 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.

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

3.5.6 *Stability of slopes underlain by mudrocks*

As discussed in Chapter 2, Section 2.6, slopes underlain by mudrocks commonly show evidence of past instability, even when the slope angles are small (e.g. 10 to 15 degrees). This is not really surprising when we consider that all of the characteristics described in Sections 3.5.2 to 3.5.5 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 & 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.

A useful review of experience with unstable slopes on shales, and on shales with interbedded sandstones, is given by Deere & Patton (1971). 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.

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 2.6.

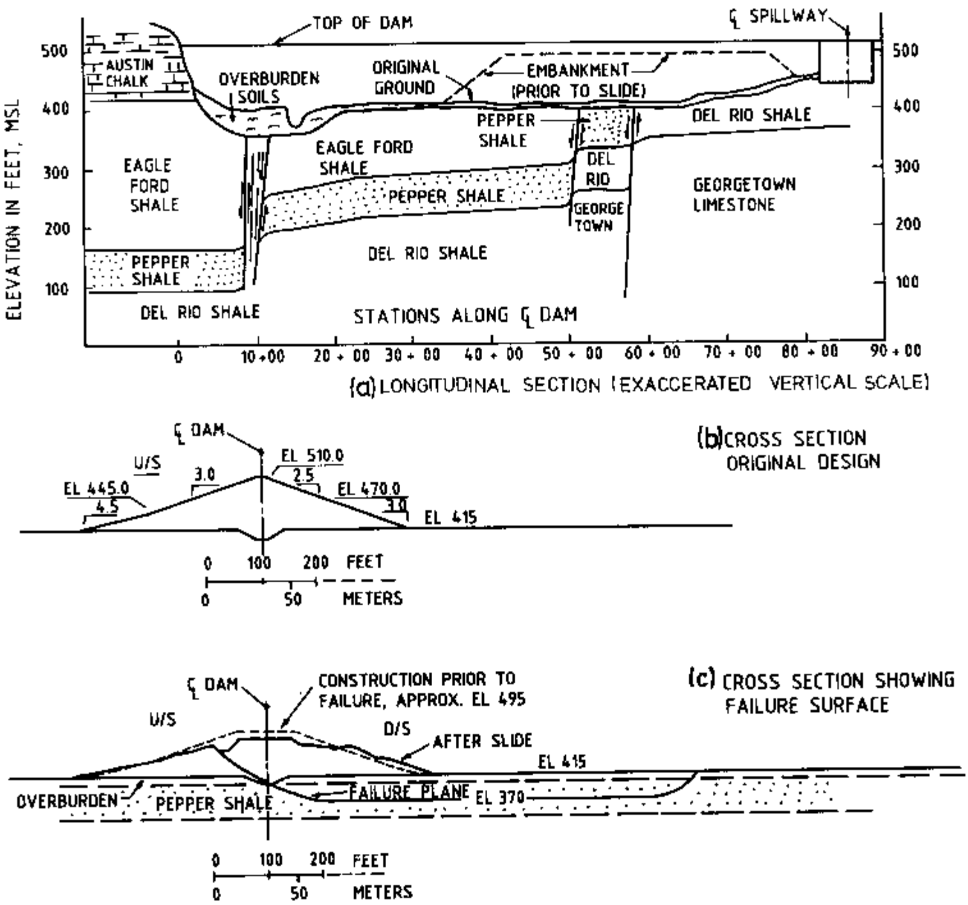


Figure 3.20. Longitudinal and cross sections, Waco Dam. From Stroman, Beene & Hull (1984).

3.5.7 *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 folded, locally (Fig. 3.20a).

The originally designed embankment is shown in outline on Figure 3.20b. The design was based on the assumption that the weakest foundation material was a localized 12 m thick layer with unconsolidated undrained strength of ϕ = 5 degrees, 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 (Fig. 3.20a). 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 prior to the slide, and at the end of the reconstruction. Stroman et al 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 degrees, with zero cohesion. They state that: 'This is the laboratory test 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 & Peck 1967, Skempton & Hutchinson 1969, and subsequently by Skempton & 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 the 7 to 9 degree residual strength may have existed in the Pepper Shale, before the slide, and have 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 3.5.3, such minor bedding faults can be difficult to recover and recognize in drill cores.

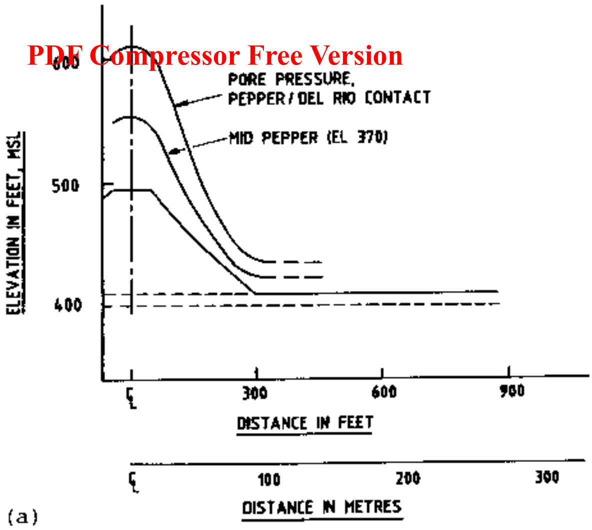
3.5.8 *Suitability of mudrocks for use as construction materials*

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, near Sydney, a trial road pavement built using ripped and trafficked shale has performed satisfactorilly for 20 years (Won 1985, Minty & Smith 1980). The shale is weak to medium strong. Re-sampling and testing of the material showed that there had been little breakdown in service.

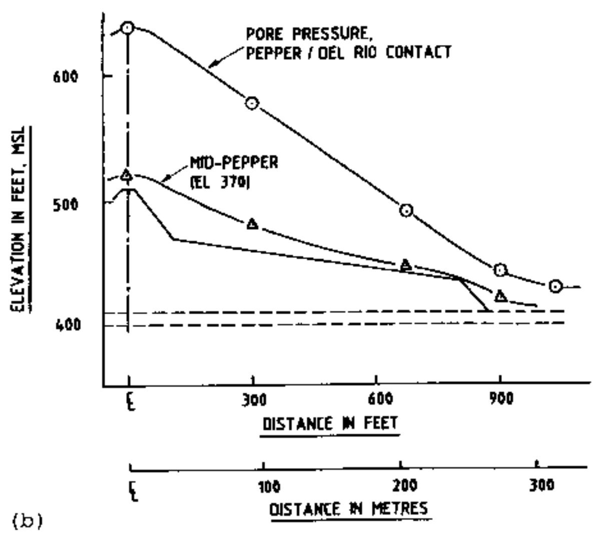
Siltstones in the medium strong to very strong range have been used successfully as rockfill for some years. Excavation of test pits into old spoil dumps comprising these materials has shown that slaking does not occur in the rock below the near-surface two metres, ie. in a constant humidity environment.

To the knowledge of the authors, no mudrocks have been used successfully as rip-rap.

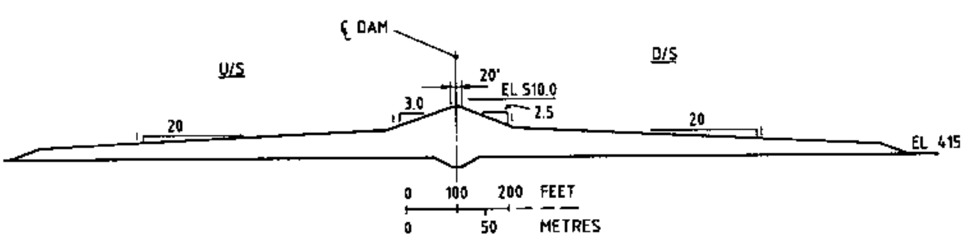
Random fills, earthfills and cores for embankment dams have been built successfully using mudrocks in various conditions ranging from fresh to extremely weathered.



(a)



(b)



(c)

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

3.5.9 *Mudrocks – Checklist of questions*

– Slaking or disintegration on exposure?
– Swelling on exposure?
– Valley bulging?
– Soluble minerals in beds or veins?
– Slickensided fissures?
– Progressive shear failure?
– Bedding surface faults or shears?
– 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 similar ranges of compositions 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).

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

Rock name	Particle shapes, grading	Minerals	
		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% felspar; 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

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 by James (1977), James & Lupton (1978) and James & Kirkpatrick (1980).

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. 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 & 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 percent 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 & 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 2.24.

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 occur only 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 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 these layers to be greatly increased, near the surface.

Where whole hillsides are underlain entirely by rocks of the sandstone group, these rocks

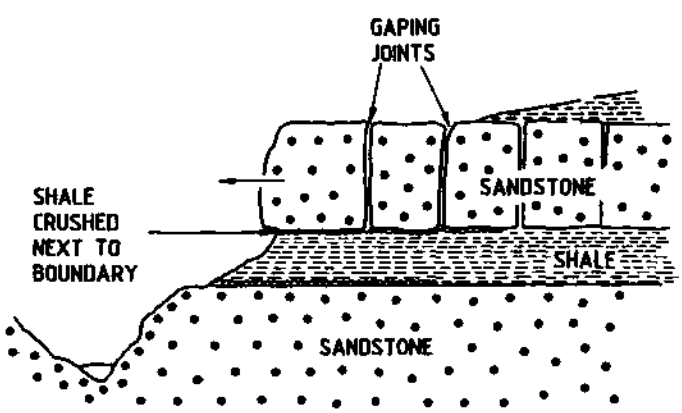


Figure 3.22. Commonly observed features close to cliffs formed by horizontally bedded sandstones with shale or siltstone interbeds. Based partly on Deere & Patton (1971).

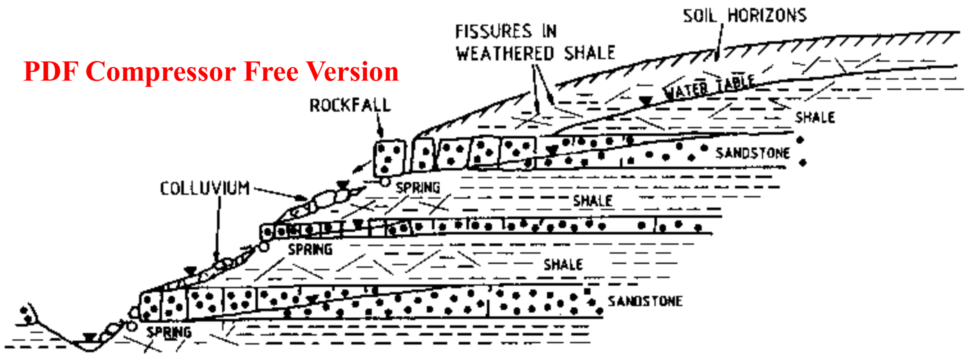


Figure 3.23. Collapse of outcropping sandstone beds due to removal of support of undertying shales. Based on Figure 13b of Deere & Patton (1971).

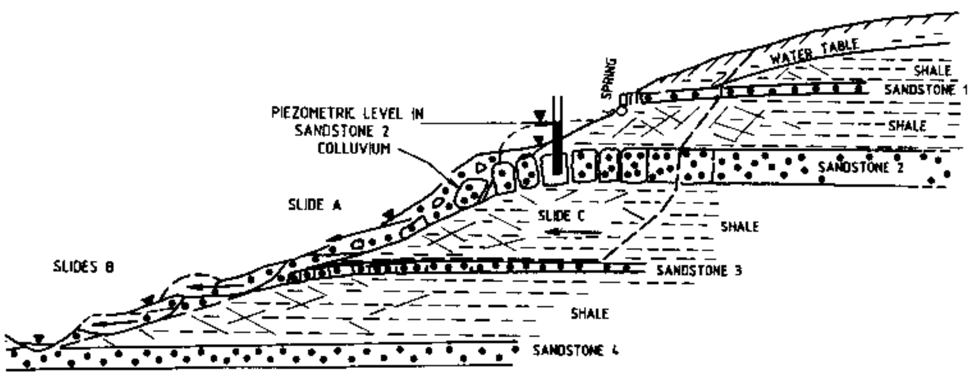


Figure 3.24. Landslides which can occur in a slope where sandstone beds are covered with colluvium. Based on Figure 13c of Deere & Patton (1971).

commonly form steep slopes or cliffs. Slope failures are rare and usually 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 13 of Deere & 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 meta-stable top of the slope below, causing Slide No. 2 (Deere & Patton 1971).

Deere & Patton (1971) point out that deep-seated 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, Sections 2.6.3.4 and 2.6.3.5.

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 and/or dolomite in their substance fabrics. They include sedimentary carbonate rocks, marble (metamorphosed carbonate rock) and calc-silicate rocks formed by the metamorphism of impure carbonate rock. Table 3.2, proposed by Dearman (1981), is a practical engineering classification for sedimentary carbonate rocks.

Sedimentary carbonate rocks range widely in strength when fresh (unweathered). Dense, fine grained limestones and dolomites are commonly non-porous and very strong or extremely strong. At the other extreme, many calcarenites comprise loosely packed, weakly cemented shell fragments, and these rocks are highly porous and weak to extremely weak (Table 2.4).

Marble, which comprises coarsely crystalline calcite, is usually dense, non-porous, and strong or very strong. Calc-silicate rocks contain carbonate minerals together with other min-

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

erals often including olivine, diopside and garnet. Their physical properties are usually similar to those of marble.

Cruden & Hu (1988) and Dearman (1981) present data on the physical properties of carbonate rocks.

3.7.1 *Solution effects*

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 water containing dissolved carbon dioxide, and their rates of solution are very high (James & Kirkpatrick 1980, James 1981).

The term karst is used to describe terrain underlain by cavernous rocks (including carbonates and evaporites) and often also to describe the cavernous rocks themselves.

The factors involved in the formation of karst are essentially the same as those which contribute to the development of weathered profiles. These factors and their influence on weathering (including solution) have been discussed briefly in Chapter 2, Section 2.2.4.

The development, distribution and surface and subsurface structure of karst have been studied and described in much more detail than the equivalent aspects of weathered, non-soluble rocks. This is due to the challenges karst poses to water resource development, and the interest provided by its caves and unique topographic forms, to speliologists and geomorphologists. Detailed discussion of karst development, structure and hydrogeology can be found in Sweeting (1972, 1981) Milanovic (1981), Bonacci (1987) and Ford & Williams (1989).

Exploration of karst, in particular the location of cavities, is very difficult. Geophysical methods appear promising. Details of methods and experiences can be found in Cooper & Ballard (1988), Handfelt & Attwooll (1988), Sarman (1983), Ballard et al. (1983), Bjelm et al. (1983) and Gilboy (1987).

3.7.2 *Main carbonate rock types – Solution effects and weathering profiles*

The following solution effects are observed commonly in the main carbonate rock types.

3.7.2.1 *Rock masses composed of dense, fine grained limestone or dolomite comprising more than 90 percent carbonate*

When fresh and intact these dense, fine grained rocks usually have very low porosities and their permeabilities are effectively zero. Groundwater infiltration and flow is therefore confined almost entirely to joints or other defects, and a weathered profile of the type shown on Figure 3.25 is commonly developed.

The main feature of this profile is the virtual absence of any weathered rock substance. This is because dissolution of the limestone substance leaves less than 10 percent of insolubles, which infill some cavities and form the residual soils at and near the surface.

The upper surface of carbonate rocks affected by solution often consists of pinnacles of fresh rock separated by deep slots and cavities which may be partly or wholly filled with residual soils. Figures 3.26 and 3.27 show examples of this.

3.7.2.2 *Rock masses composed of dense, fine grained rocks containing 10 to 90 percent carbonate*

In these rocks the pattern of solution cavities follows the defect pattern, as in Figure 3.25, but

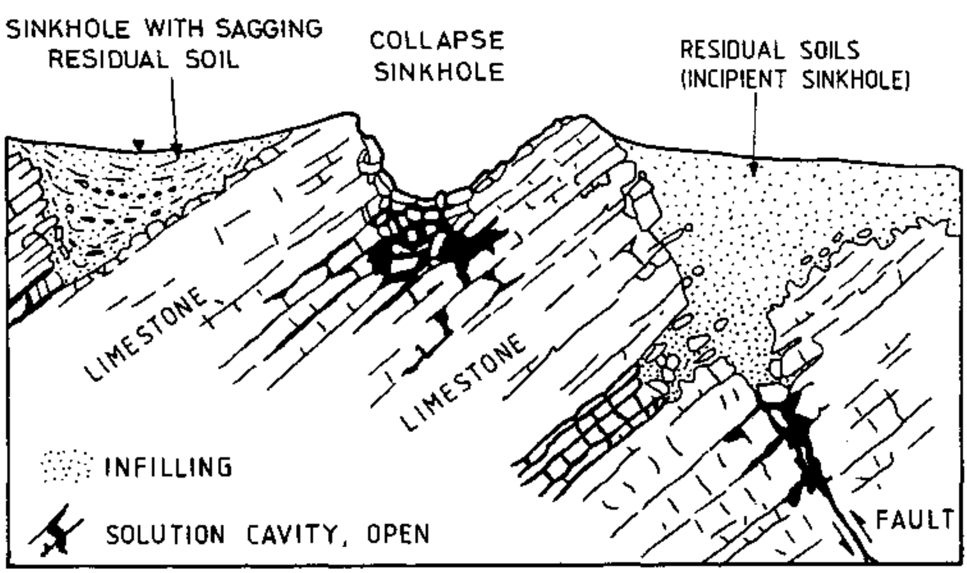


Figure 3.25. A typical weathering profile in dense, relatively pure carbonate rock. (Based on Deere & Patton 1971, Fig. 9)

Figure 3.26. Pinnacled upper surface of limestone, exposed in 20 m high quarry face. Photograph courtesy of Dr R. Twidale.

the rock substance next to the cavities is weathered. This weathered rock has lower density than the fresh rock due to the partial removal of carbonate. In general the lower the percentage of carbonate in the fresh rock, the higher the proportion of weathered rock present, compared to cavities, and the higher the ratio of infilled to open cavities.

3.7.2.3 *Rock masses composed of highly porous, low density rock substance*

Most carbonates which are highly porous have substance permeabilities high enough to allow groundwaters to percolate through the substance fabric as well as through defects in the mass. It is clear that these rocks are likely to be far more susceptible to solution than the dense varieties, assuming comparable fracture patterns and weathering histories (climate and duration). Experience shows this to be so – in Papua New Guinea the authors have observed that karst areas underlain by porous limestones show much more abundant and larger sinkholes than those underlain by dense limestones. In some porous limestone areas the ground surface is like a large-scale honeycomb; sinkholes from 50 m to hundreds of metres in diameter are separated by narrow ridges and pinnacles of limestone. Much of the ground surface near the landslide shown on Figure 3.36 is like this.

Figure 3.27. Closeup view of limestone pinnacles.

Weathering of the porous rocks commonly causes the development of pinnacles and near-vertical solution tubes near the surface, and extensive caves at depth often following no obvious structural defects. Figure 3.28 shows pinnacles and solution tubes and the resulting extremely irregular soil-rock interface on porous calcarenite of Pleistocene age, near Perth, Western Australia. Layers of dense stronger rock usually form laminated linings or crusts on the walls of the tubes and other cavities (Fig. 3.29). These crusts are formed when carbonate dissolved from elsewhere in the rock mass is redeposited against the walls and into voids in the adjacent porous rock.

3.7.2.4 *Properties of weathered carbonate substances*

As stated in Section 3.7.2.2, dense carbonate rocks containing significant proportions of insoluble particles (usually quartz, clay and iron oxides) are commonly found in the weathered state, due to partial removal of carbonate. Complete removal of the carbonate in most cases leaves a residual soil and a cavity. However, where the insolubles are in sufficient quantity and

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

are sufficiently well bonded by non-soluble cements, they form very or extremely weak rocks when the carbonate material has been removed completely.

Weathered materials in this latter category occur at Little Para dam and elsewhere in South Australia. The original fresh rocks are dolomitic siltstones or slates, very strong, blue-grey with SG of about 2.7. Where weathered just to the extent that no carbonate remains, they are very to extremely weak, porous materials with SG ranging from 1.8 to 2.0. They show a brittle, biscuit-like behaviour when broken. Deere & Patton (1971) and Roberts (1970) provide further descriptions of weathered profiles in carbonate rocks.

3.7.3 *Significance of solution effects*

3.7.3.1 *Need for treatment of dam foundations and reservoirs to render them watertight*

Many dams have been built successfully on sites underlain by carbonate rocks, despite the fact that solution cavities have been present at the majority of the sites. 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 clayey infilling materials in cavities presents a problem when grouting is the proposed method of treatment. Although the clay prevents grout penetration and is not readily removed by flushing between boreholes, it may be flushed out by seepage waters later when the dam is filled.

High pressure grouting designed to cause hydraulic fracturing of the infill materials has been shown by Zhang & Huo (1982) to give significant improvement in their resistance to leakage

Figure 3.29. Partly infilled solution tube in very weak, porous calcarenite, showing crust of dense, strong limestone

and piping. This method has since been used successfully at Bjelke-Peterson Dam in Queensland (Eadie 1986, McMahon 1986).

At sites where cavities are numerous and large and partly or wholly filled with clays and or/coarser grained soils, cement grouting alone is not usually relied on to form a cutoff. The other methods (mining and backfilling with concrete and construction of concrete diaphragm walls) are used. These methods were employed successfully, together with cement grouting, to construct a curtain 3.5 km long and up to 200 m deep beneath the right abutment ridge at Khao Laem Dam in Eastern Thailand (Lek et al. 1982, Somkuan & Coles 1985). This curtain was needed because the ridge was formed entirely by cavernous limestone, and the water table within it was below the proposed storage level.

Merritt (1988) describes the design and construction of an extensive grout curtain at and upstream from the 238 m high El Cajon Dam in Honduras. The double curvature arch dam is built on cavernous Cretaceous limestone in a narrow gorge. About 200 m upstream from the dam the valley becomes wider and the limestone is overlain and effectively blanketted by basaltic flows and tuffs.

The limestone was shown to be cavernous for at least 200 m into the valley sides and for at least 180 m beneath the floor, and the water table was almost horizontal at river level. In this situation it was not certain that a conventional grout curtain near the dam would achieve closure against the high water level of the reservoir. Because of this the curtain adopted was in the form of a continuous trough, extending from the dam to the basaltic rocks and beneath the floor and both sides of the gorge up to reservoir level. Construction involved more than 14 km of galleries, 514 000 m of holes drilled and grouted, 83 700 tonnes of cement and 14 930 m^3 of backfill concrete.

3.7.3.2 *Dams which have experienced significant leakage problems*

Table 3.3 lists dams which have recorded very large leakage rates during operation, and Table 3.4 lists others where the reservoir has never filled because leakage rates exceeded the inflow. Three dams which have recorded significant leakages are discussed below.

Keban Dam. At this 200 m high, 1100 m long dam in Turkey very large caves were found in fractured, metamorphosed carbonate rocks, both during construction and during first filling in 1976. Bozovic et al. (1981) outline the extensive foundation treatment carried out during and after construction. The construction stage work included the following:

– 11 km of galleries ;

– 178 000m^3 of backfill concrete, placed in cavitites, replacing excavated 'gouge,' and forming large sections of diaphragm walls;

– grouted holes totalling 338 300 m in length;

– 135 200 tonnes of injected solids.

Near the end of construction a large cave discovered more than 250 m below the dam was treated by injecting 64 000 m of concrete and other solids.

Table 3.3. Dams at which very high leakage rates were recorded (Erguvanli 1979).

Dam	Country	Rock	Maximum recorded leakage (m^3/s)
Hales Bar	USA	Carboniferous limestone and shale	54
Great Falls	USA	Limestone	13
Camarasa	Spain	Jurassic dolomite limestone	12
Keban	Turkey	Palaeozoic limestone	7
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 Neogene limestone
Montejagne	Spain	Jurassic limestone
Perdikas	Greece	Miocene limestone
Villette Berra	Italy	–

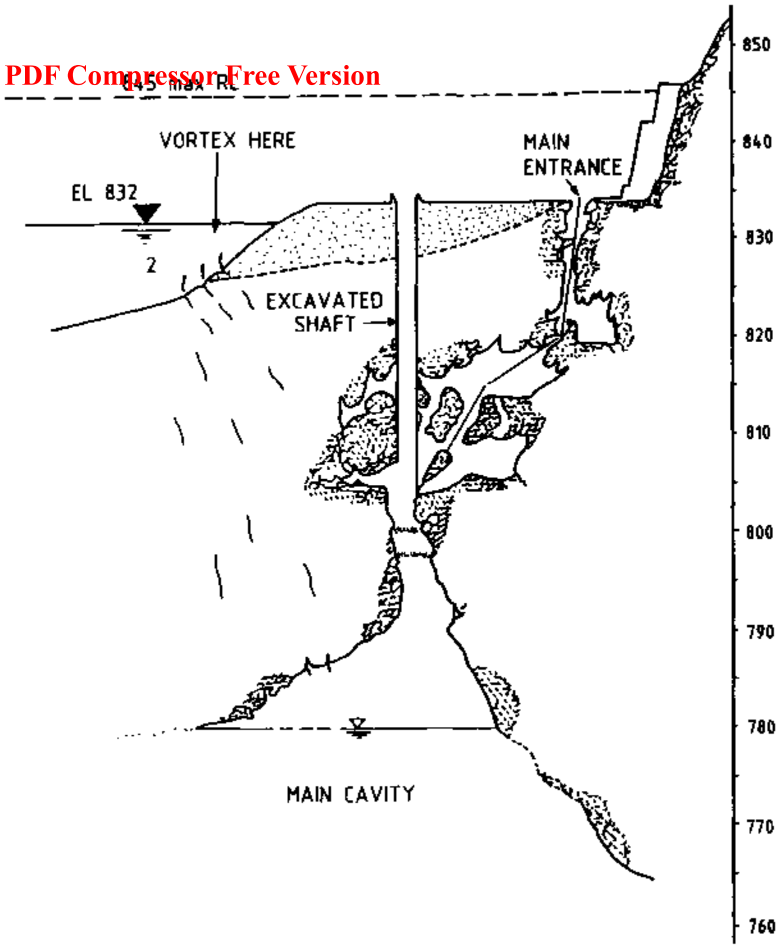


Figure 3.30. Petek Cavity, beneath the right shoulder of Keban Dam (Bozovic et al. 1981).

The much larger Petek Cavity, found during reservoir filling in 1976, is shown on Figure 3.30. When the reservoir reached El 832 vortices appeared above the cave area as shown, and the flow rate of springs downstream from the dam increased from 7.5 to 26m^3/s. A direct connection from the reservoir via the cave was inferred.

Exploration of this feature included diamond drill holes, the shaft shown on Figure 3.30, direct access and visual inspection, and echo sounding of the submerged part. It was concluded that the cave is at least 90 m long, 30 m wide and 50 m high and that rock above the cave was of poor quality and potentially unstable.

Remedial works included exposure and dental treatment with concrete of the rock at the ground surface above and near the cave, grouting of the weak rock around and above it (1385 m of holes, 2900 tonnes of solids consumed) and backfilling the cavity with 605 000 m^3 of rock, gravel, sand and clay.

After completion of this treatment the spring discharge was reduced to 8.69 m^3/s, with the reservoir close to design level. Bozovic et al. conclude that the leakage has stabilised at this

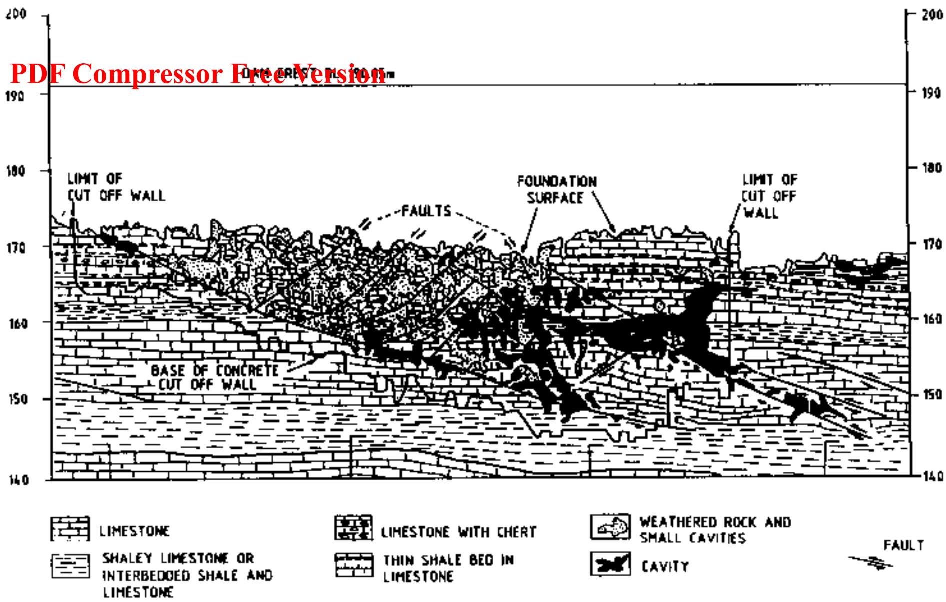


Figure 3.31. Geological section along part of upstream face of Hales Bar Dam (from Burwell & Moneymaker 1950)

figure, which is negligible compared to the inflow to the reservoir. They note however that monitoring is necessary to detect any future changes.

Hales Bar Dam. Probably the longest struggle to control leakage through solution cavities occurred between 1913 and 1963 at Hales Bar Dam in Tennessee. The concrete gravity section of the dam, 34 m high and 700 m long, was founded on gently dipping dense, strong limestones with interbeds of calcareous shales and chert. Extensive zones of cavities and weathered rock were found during construction and were treated by cement grouting.

Figure 3.31 shows a section through one of the two main leakage zones. Leakages became evident immediately after filling, and continued at varying rates (up to 54 m^3/s) for 50 years, despite various treatments including blanketting, asphaltic grouting and contiguous concrete pile curtains (Burwell & Moneymaker 1950, Tennessee Valley Authority 1963). The dam was abandoned in 1963 and replaced by a new structure downstream.

Congateringa Dam. Even if carbonate rock forms only a small part of the foundation of a low dam it can allow significant leakage if it is cavernous and not treated. At the 13 m high Congateringa (earth) Dam in South Australia (Fig. 3.32) four limestone beds each less than 50 mm thick occur within tightly jointed schist and quartzite. These beds were exposed in the cutoff trench just below Full Supply Level and showed no evidence of any cavities.

Because of the high quality of the rock and the low head it was decided that a grout curtain was not required. However the floor of the cutoff trench was sealed with pneumatically applied mortar to protect the clay core from possible erosion by water seeping along joints. When the reservoir was filled many small water flows and seepages issued from the outcrops of the limestone beds downstream (Fig. 3.32). These flows were estimated to total 700 m^3/day, significant to the project and more than anticipated bearing in mind the general good quality of the rock.

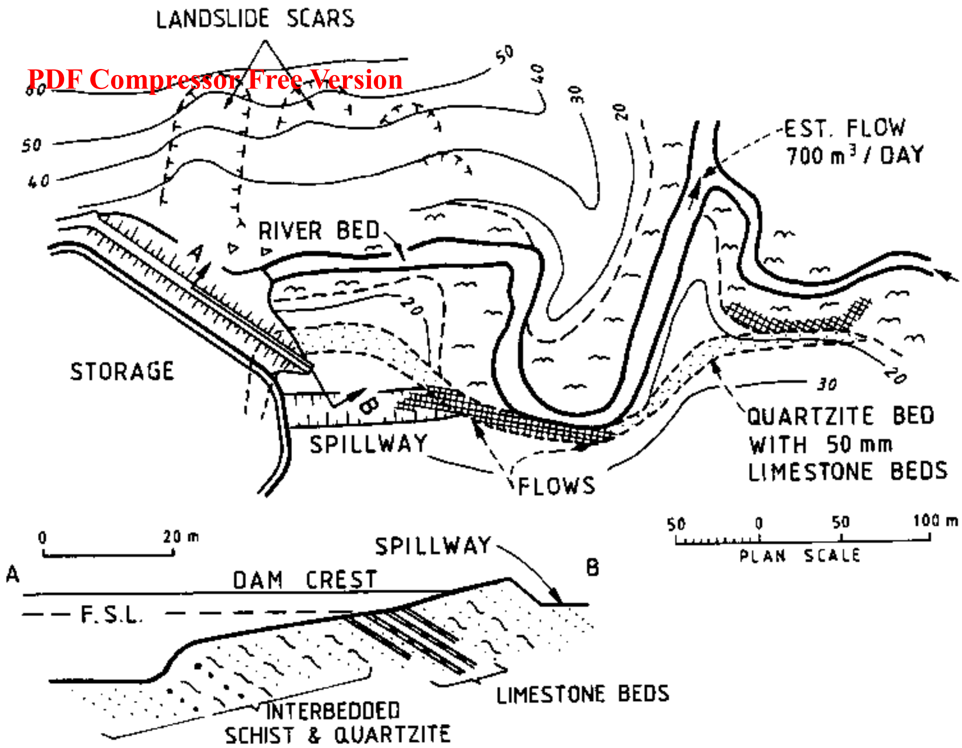


Figure 3.32. Plan and cross section of part of Congateringa Dam.

3.7.3.3 *Low strength and erodibility of some weathered carbonate materials*

At Hales Bar Dam (Fig. 3.31) much of the material in the cavernous zones was described as 'rotten rock' (Burwell & Moneymaker 1950). It is clear, and this has been assumed on Figure 3.31, that this material is weathered carbonate rock. As discussed in 3.7.2.4, such material may be very to extremely weak, friable and erodible. It is considered likely that some of the deterioration which occurred over 50 years in this foundation was due to erosion of such materials.

Dolomitic siltstone is the predominant foundation rock at Little Para concrete decked dam (Fig. 3.33). Beneath the upper right bank there is a 1 to 3 m wide zone of very weak, weathered, low density dolomitic siltstone grading into extremely weathered siltstone (slightly dispersive clayey silt) which contains cavities. The zone is parallel to the plinth and 6 to 12 m below it, and daylights near the downstream toe of the dam where it was exposed by trenching during the site investigation. The possibility of erosion of the clayey silt and extremely weak rock during dam operation was considered. The zone was left in place because it was mainly beneath the upper part of the abutment where it was considered to be adequately bridged by strong siltstone and quartzite. The zone was carefully cement grouted by the downstage method (Boucaut & Beal 1984). This treatment was successful, as evidenced by the lack of seepage during 12 years of reservoir operation.

3.7.3.4 *Possible collapse of cavities*

In areas underlain by cavernous carbonate rocks collapse of the ground surface into a pre-

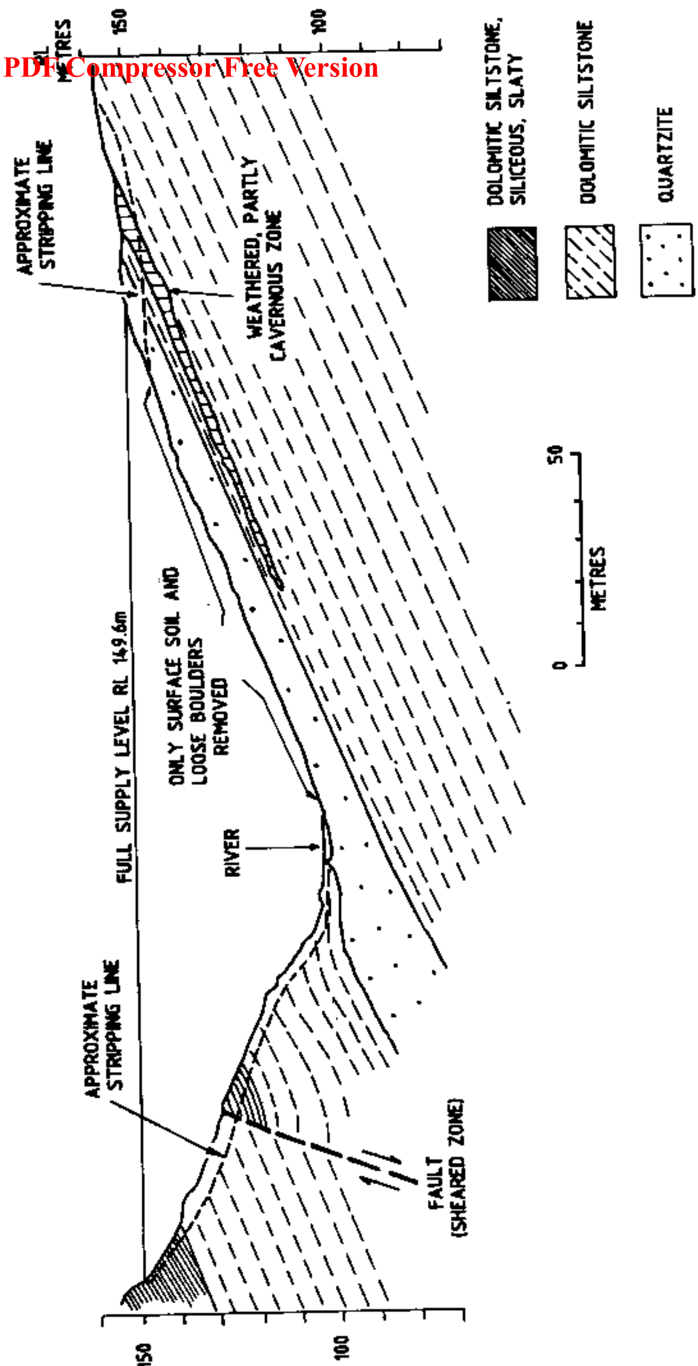


Figure 3.33. Little Para Dam, foundation geology (based on Boucaut & Beale 1984).

viously unsuspected cavity is a relatively common occurrence. Such collapses or 'sinkholes' occasionally happen naturally but usually they are induced or accelerated by man's activities. Mechanisms for sinkhole formation are described in detail by Newton & Tanner (1987) and Wilson & Beck (1988) and include the following:

– dewatering, which can cause loss of buoyant support for the rock or soil forming the roof of the cavity, or by steepening the hydraulic gradient, cause erosion and cavitation of overlying soils, into the cavity;

– inundation or wetting up previously dry soil bridging over a cavity, giving rise to piping or to its collapse due to loss of cohesion, loading by construction plant, embankments etc.;

– vibrations from machinery or blasting.

It is clear that for sites located in karst areas, there will be a risk of cavity collapse affecting any part of the project works, including borrow areas and haul roads as well as the dam embankment and associated structures. The worst imaginable event would be the collapse of a cavity beneath a dam embankment, during reservoir operation.

Drumm et al. (1990) describe a theoretical approach to the prediction of the stability of residual soil overlying a cavity. Their model neglects the effects of groundwater and because of this would not be applicable in the vicinity of dams.

3.7.3.5 *Dewatering of excavations*

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.

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

It is considered likely that the leakage at Hales Bar Dam, which continued and increased despite the various treatments, occurred because of erosion of weak, weathered rock and of clay fillings in cavities, and some continuing dissolution of limestone. Frink (1945) inferred that the maximum rate of limestone solution in the leakage zones was about 2.5 mm/year, based on measurements of the recession of limestone surfaces in calyx holes which remained open at about 20 m depth for more than a year. Frink noted that the surfaces measured were below a cavity 300 to 1000 mm across, showing 'moderate' water velocity.

James & 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:

1) the solubility (c) of the mineral, which is the amount which can be dissolved in a given quantity of solvent, at equilibrium, and

2) 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 in it.

James & Lupton (1978) point out that for water flowing through a joint in effectively impervious but soluble rock substance, widening of the joint by solution of the rock walls will

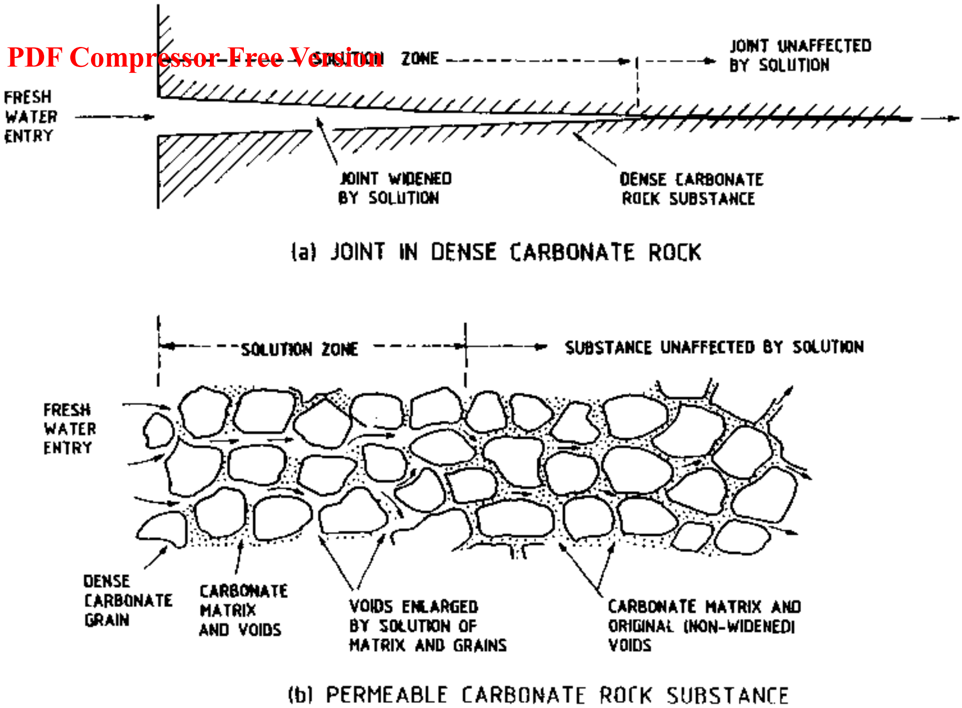


Figure 3.34. Development of solution zones in a) a joint and b) soluble rock substance. (Based on James & Lupton 1978).

occur only until the water becomes saturated with the soluble material. Thus a solution zone (Fig. 3.34a) 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 (Fig. 3.34b). James & Kirkpatrick (1980) use the methods and models of James & 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 equation to predict the ways in which calcite dissolves in joints in the ground:

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

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. Mathematical modelling provides predictions of the way joints enlarge as water flows through them. Their Figure 3, reproduced here as Figure 3.35, shows their predictions for the enlargement of limestone joints with initial apertures ranging from 0.5 to 2 mm, by pure water.

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

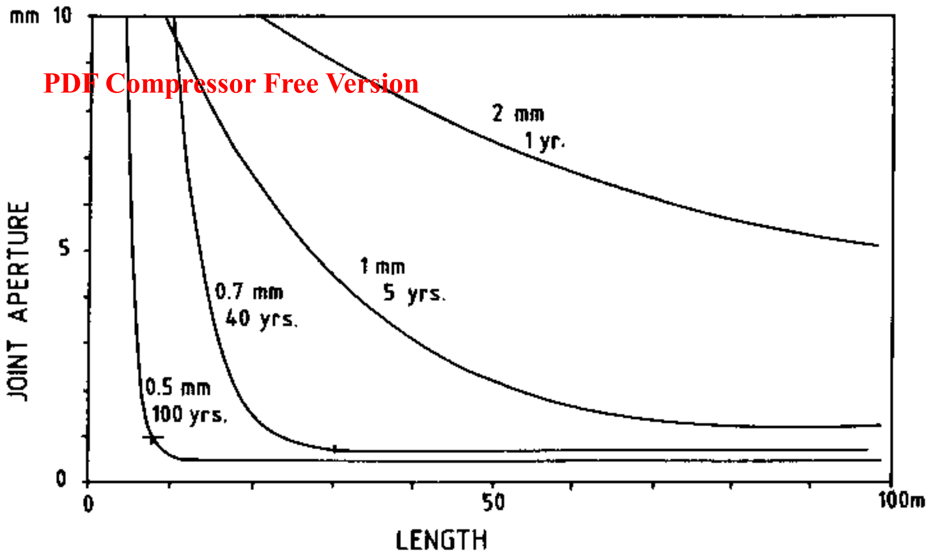


Figure 3.35. Enlargement of joints in calcium carbonate rock by pure flowing water (James & Kirkpatrick 1980, reproduced by permission of the Geological Society).

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 & 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 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.

James & Kirkpatrick also discuss solution rates in unjointed rock substance which is either all soluble or containing particles of soluble minerals, and with intergranular permeability. Again they use the theoretical models of James & Lupton (1978). They show that the length of the solution zone (Fig. 3.34b) depends mainly on the solution rate constant and the solubility, and that other site factors such as the rock fabric, the size and shape of veinlets or the proportion of soluble mineral 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 solubility, the proportion of soluble mineral present again being less critical.

Table 6 of James & Kirkpatrick (1980), reproduced here as Table 3.5, shows the calculated limiting seepage velocities which would cause 0.1 m/yr downstream migration of solution zones in permeable rock or soil substances containing limestone or evaporite minerals. It can be seen that the calculated seepage velocity for limestone is higher than would normally be tolerated in a dam foundation. James & Kirkpatrick conclude that control of seepage by a good grouting programme or other means should prevent dangerous progressive solution of these types of carbonate substances.

James (1981) carried out laboratory studies on a range of carbonate rocks of different origin

Table 3.5. Solution of soil or rock substance with intergranular permeability (James & Kirkpatrick 1980).

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

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

types and compositions and showed that in pure water their solution rate constants were similar and their solubilities were virtually the same as that of pure calcium carbonate. He showed also for one sample that small amounts of carbon dioxide dissolved in the water lowered the solution rate by a factor of about 10, but caused larger increases in the solubility. He concluded that assessment of the potential for progressive solution of a dam foundation required determination of the following:

– the chemical composition of the inflowing seepage water,

– the sizes and distribution of open joints.

He concluded also that joints with apertures less than about 0.4 mm were 'unlikely to be dangerous in most foundations in carbonate rocks,' thus confirming one of the the main conclusions of James & Kirkpatrick (1980).

3.7.5 *Altered carbonate rocks*

At the site for Hinze rockfill dam in Queensland, scattered outcrops of quartzite occurred over the upper right bank. The quartzite appeared to be in several steeply dipping beds. The outcrops had a pitted appearance and contained what appeared to be irregular solution cavities.

During raising of the dam in 1987 grouting operations and exploratory diamond drilling showed that the quartzite had been formed from dolomite and limestone by a process of silicification, that is, replacement of the carbonate minerals by silica. The degree of silicification appeared to have been about 75 percent near the surface, and decreased with depth. Within about 40 m of the surface the partly silicified material contained many clay-infilled and some open cavities; at greater depths exploration showed the cavities to decrease in size and number.

From the distribution and nature of the 'quartzite' it seems likely that the silica was released from adjacent metamorphic rocks during tropical lateritic weathering, and deposited as 'silcrete' in convenient pore-space provided by the rapidly dissolving carbonate rocks (see Fig. 3.42).

These unexpected conditions were allowed for by changing the single row to a three-row grout curtain, blanketting the upstream outcrop area of the 'quartzite', and construction of a drainage trench beneath the downstream toe of the embankment.

3.7.6 *Suitability of carbonate rocks as construction materials*

Dense carbonate rocks in the strong to extremely strong range are used extensively for the production of aggregates for concrete, bituminous concrete and pavements. Generally they perform very well.

However, a few dolomitic rocks have been found to react with alkalies in Portland cement, giving rise to expansion and cracking of concrete, similar to the effects of alkali- silica reaction. Also, many limestones contain inclusions of chert which is a form of silica and often reactive.

Hugenberg (1987) describes the results of investigations at the 38-year old Center Hill Dam, in Tennessee, where concrete expansion had caused binding of the spillway gates and closing of expansion joints in bridge spans. It was found that these effects were caused by reaction of some of the aggregate (argillaceous dolomitic limestone of Ordovician age) with alkalies in the cement.

Luke (1963) records similar alkali-carbonate reaction effects at the powerhouse of Chickamauga Dam in Tennessee.

Other examples are given in Highway Research Board (1964).

Guillott (1975, 1986) describes petrographic observations of reactive carbonate rocks and discusses the processes believed to cause the concrete to expand. He concludes that only very fine grained dolomitic rocks containing some clay are likely to cause expansion by alkali-carbonate reaction.

Because of the above it is advisable to check for the possibility of reactivity, when considering the use of very fine grained carbonate rocks together with high-alkali cement. This checking should include petrographic examination (American Society for Testing and Materials 1974a) and testing by the chemical method (American Society for Testing and Materials 1974b) or Standards Association of Australia (1974a) and the rock cylinder method (American Society for Testing and Materials (1974d).

Carbonate rocks are widely and successfully used as rockfill, random fill and rip-rap. However, some argillaceous or shaley limestones develop fine cracks and may eventually disintegrate, on exposure. Such rocks would not normally be acceptable for use as rip-rap. These rocks can be considered as calcareous mudrocks and their deterioration occurs as described in Chapter 2, Section 2.4.1.

Carbonate rocks are unsuitable for use in filter zones, because of their susceptibility to dissolution and cementation.

3.7.7 *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 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 3.36 to 3.38 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 (Fig. 3.38). 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 Vajont landslide, Hendron & Patton (1985) have shown that clay-rich units and clay are present along and near most of the failure surface.

Figure 3.36. Air photo showing the scar of an old landslide in folded limestone, near the Mubi River, Papua New Guinea.

James (1983) draws attention to situations in Sri Lanka where solution of near-horizontal limestone near valley floors has caused collapse, undercutting and landsliding in overlying beds. Retreat of these beds has produced broad valleys bounded by steep escarpments, with hanging tributory valleys. James considers these almost 'glacial-like' valleys to be useful indicators of the presence of limestone, in areas devoid of outcrop.

3.7.8 *Carbonate rocks – Checklist of questions*

– Cavities, air-filled or water-filled?
– Cavities, soil-filled?
– Collapse of cavities?
– Extremely irregular, often pinnacled, surface of fresh rock

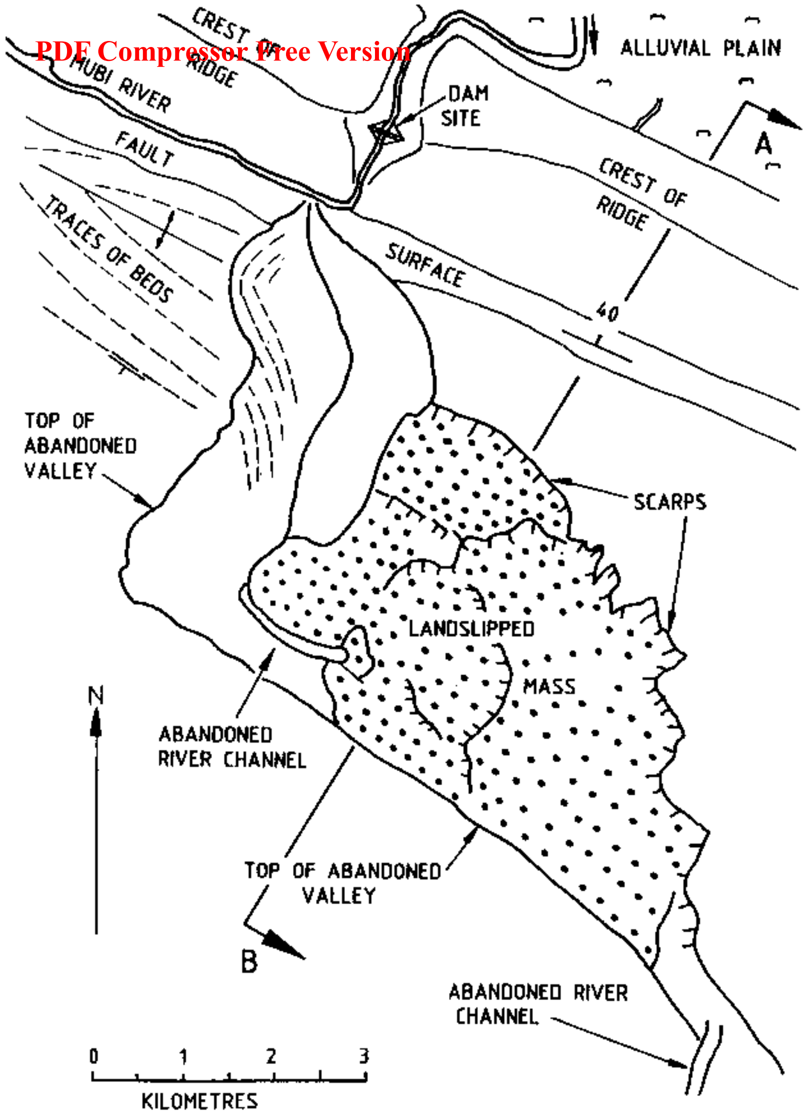


Figure 3.37. Geological plan of the area shown on Figure 3.36.

– 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?
– Potential for continuing solution during operation of dam?

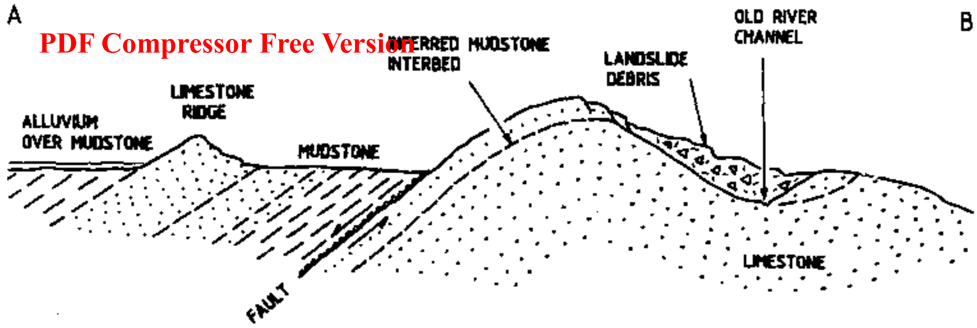


Figure 3.38. Sketch section through the area shown on Figure 3.36.

– Alkali-carbonate reaction?
– Chert present: Alkali-silica reaction?
– Shaley (argillaceous) rocks: Durability?
– Unstable slopes, where interbeds of mudrocks are present?

3.8 ALLUVIAL SOILS

3.8.1 *Occurrence and description*

For the purposes of this discussion 'alluvial soils' will include 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 3.39 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).

The main types of deposits shown on Figure 3.39 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

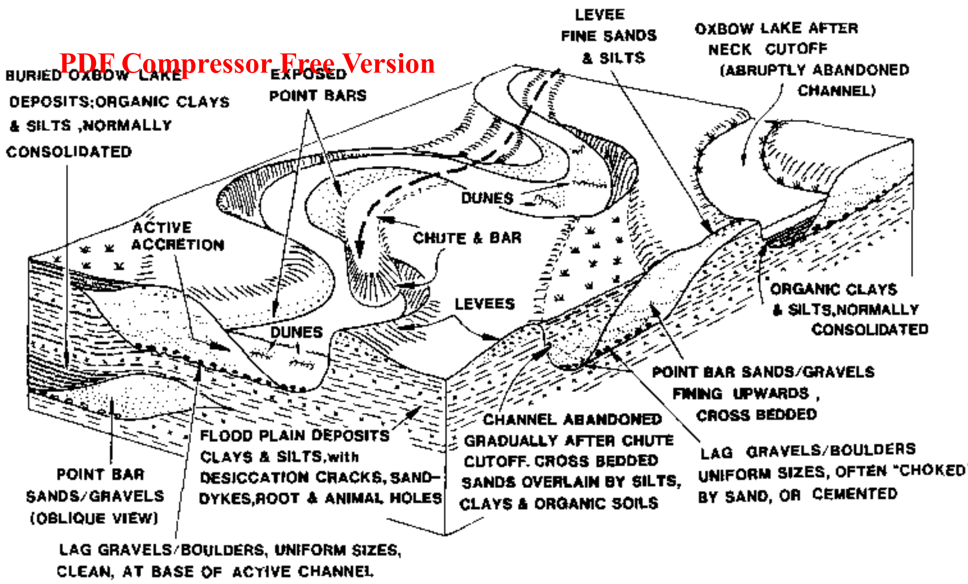


Figure 3.39. Schematic view of soils deposited by a meandering river (based partly on Fig. 1 of Walker 1984).

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-dykes 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 when parts of the meandering channel are cut off from the stream, during floods. 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 3.39 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 steam bed geometry.

Cary (1950) reports that gravel bars in several fast-flowing rivers in USA 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 USA, 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 at the downstream ends of rapidly aggrading bars, in fast-flowing rivers. 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 intermittant, 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.

3.8.2 *Properties of alluvial soils*

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.

3.8.2.1 *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 Figure 9.1. Clean gravels can be interlayered with sands or sandy gravels, giving overall horizontal permeabilities 10 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 angle in the range 28 to 35°. Deeper deposits are more likely to be dense, less compressible and have a high effective friction angle.

3.8.2.2 *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, by bottom-dump barges. The local cavitation zones which caused the sinkholes are believed to have formed when excessively high flow rates through the extremely permeable open-work gravels caused adjacent sandy layers to migrate into their large voids.

3.8.2.3 *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 & 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.

3.8.2.4 *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 or even 100 or 1000 times the vertical.

The permeability of these deposits is often increased by the desiccation cracks, sand-dykes, fissures and holes left by burrowing animals and rotted vegetation. However, 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 to 2 m. In these cases, the undrained shear strength is very low and compressibility very high, for the soils at depth.

3.8.3 *Use 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 impervious core zones include Blue Rock (Victoria), Blowering (NSW) and Bjelke-Peterson and Proserpine (Queensland).

3.8.4 *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?

3.9 COLLUVIAL SOILS

3.9.1 *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 3.40 and 3.41 show typical environments in which scree and slopewash deposits are formed.

3.9.1.1 *Scree and talus*

These are deposits of rock fragments which break off 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 comprises large

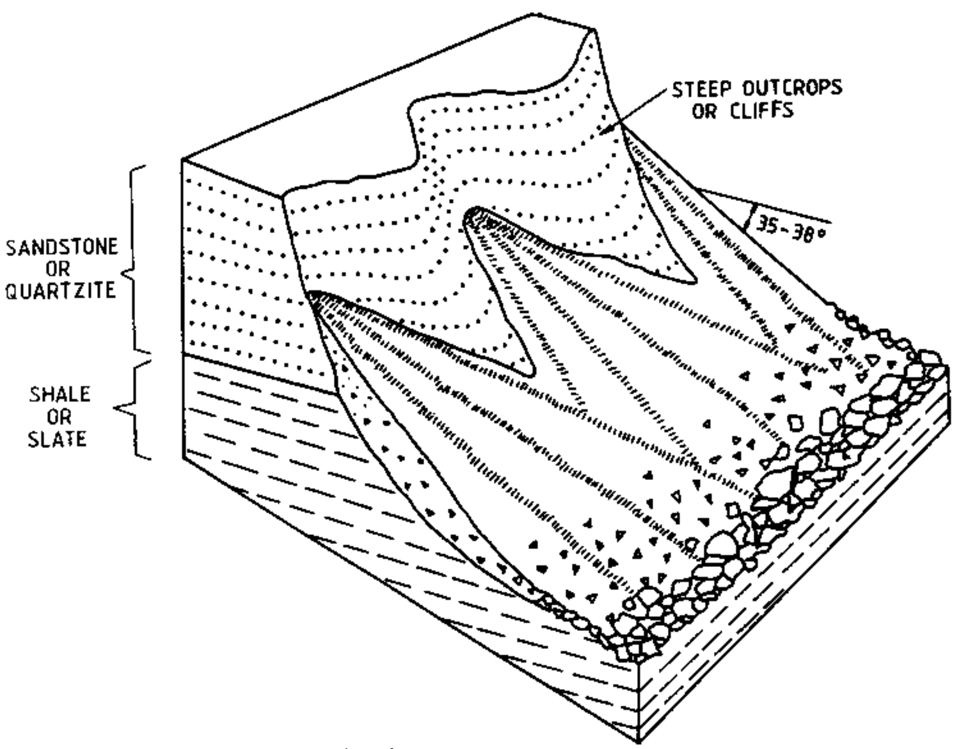


Figure 3.40. Schematic view of scree deposits.

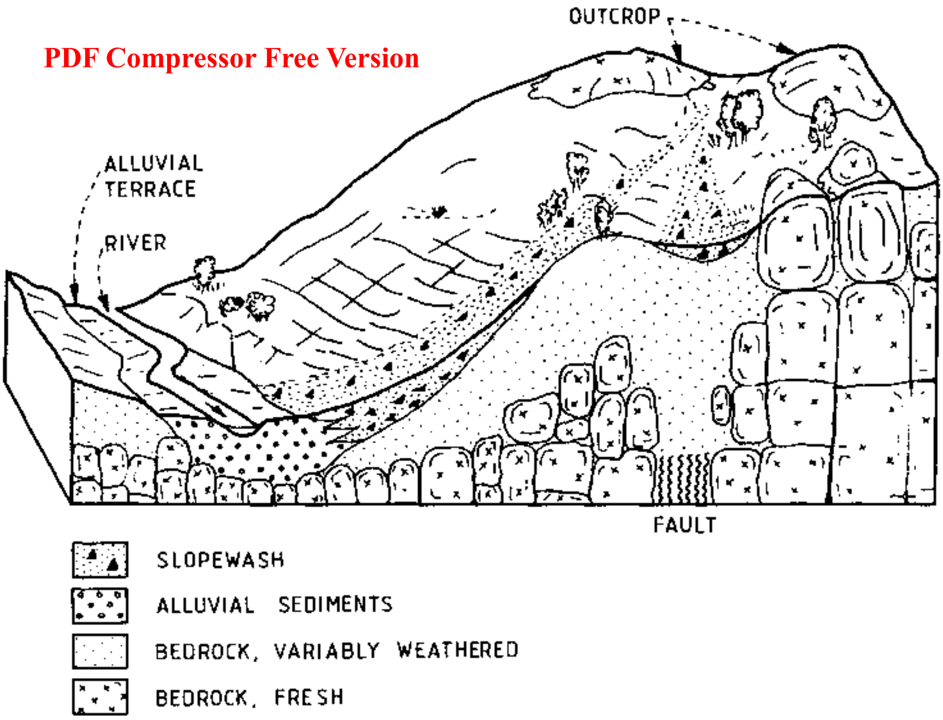


Figure 3.41. Schematic view of residual, slopewash and alluvial deposits.

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.

3.9.1.2 *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 3.41. Near the base of steep slopes slopewash soils often overlie, or are intertongued with, alluvial deposits (also shown on Fig. 3.41).

In cold climates (see Section 3.11.4) 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.

3.9.1.3 *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.

It is common to find landslide debris overlying river alluvium, at sites where landslides have dammed and diverted pre-existing rivers. This situation was found at the first site investigated for Talbingo Dam (see Section 2.6.3.1 and Fig. 2.30).

Deposits of landslide debris are often underlain by a sheared or slickensided zone (the slide surface), and there may be several surfaces of 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.

3.9.2 *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 described in the sections below.

3.9.2.1 *Scree and talus*

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

As they 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.

3.9.2.2 *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 Fig. 3.41) 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.

3.9.2.3 *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 2.6.1).

3.9.3 *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 2.6.3. The landslide debris, composed mainly of extremely weathered basalt, 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 (NSW, Australia).
– Dartmouth (Victoria, Australia).
– Thomson (Victoria, Australia).
– Trengganu (Malaysia).

Slopewash derived from extremely weathered rhyodacite was used for the core at Tuggeranong Dam (ACT, 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.

3.9.4 *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?

3.10 LATERITES AND LATERITIC WEATHERING PROFILES

3.10.1 *Definitions of laterite*

The term laterite has no single definition. As discussed by Sueoka (1985), laterites have been defined in physical (e.g. colour), chemical, morphological, agronomical and engineering terms. Townsend et al. (1982) use the following definition which covers the authors' experience of such soils in Australia and South-east Asia:

'Laterite refers to varied reddish highly weathered soils that have concentrated oxides of iron and aluminium and may contain quartz and kalolinite. 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.'

Laterites are believed to have been formed under 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 & Aleva 1990, Chapters 4 and 6). Laterites have been developed on the full range of common igneous, metamorphic and sedimentary rocks (Gidigasu 1976, Chapter 4).

3.10.2 *Composition, thicknesses and origin of lateritic weathering profiles*

Selby (1982) describes the following features of lateritic weathering profiles, some of which are shown also on Figure 3.42.

Thickness (m)	Description
0 to 2	Soil zone, often sandy and sometimes containing nodules or concretions; this may be eroded away.
2 to 10	Crust of reddish or brown hardened or slightly hardened material, with vermiform (or vermicular) structures (i.e. having tube-like cavities 20 to 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 of white clayey 'kaolinitic' material with patches of yellowish iron and aluminium sesquioxides.
Up to 60 but generally less than 25	Pallid zone of 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 Fig. 3.42).
1 to 60	Weathered margin of deeply weathered rock showing original rock structures.

The 'crust' in the above description is commonly known as ferricrete, when it is composed

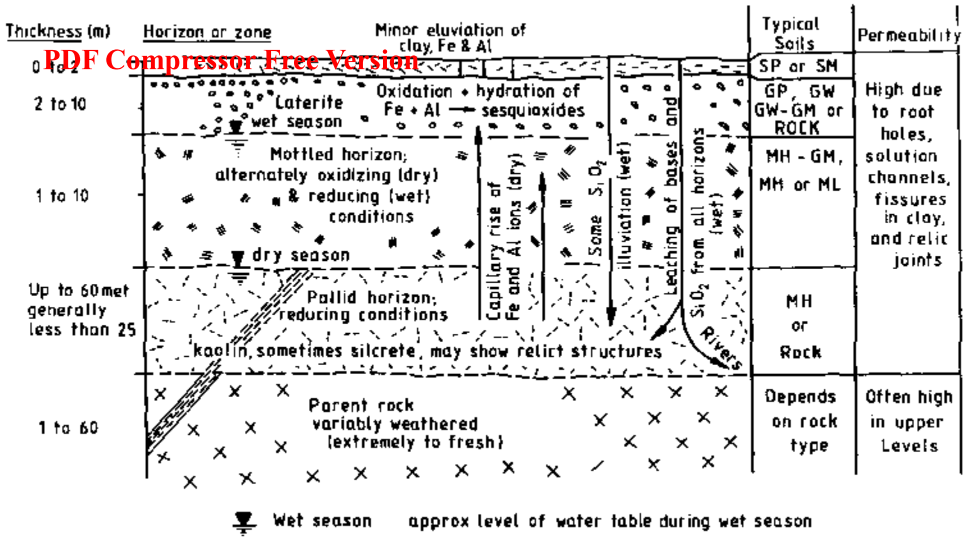


Figure 3.42. Diagram showing the lateritic weathering profile of Selby (1982) and some of the processes involved in its development (by permission of Oxford University Press).

mainly of iron oxides, and alcrete, when mainly aluminium oxides. It can be either gravel, or rock ranging from very weak to very strong, often requiring blasting for its excavation. The very weak materials often become stronger when exposed to the weather.

There are several theories about the formation of laterites (Bardossy & 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 acumulating in the upper part of the profile.

Figure 3.42 shows some of the processes thought to be involved and the resulting weathered profile.

Lateritic profiles may be much shallower than described above and shown on Figure 3.42, 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 9.2 shows common features of lateritic profiles in valley situations.

3.10.3 *Properties of laterites and lateritic profiles*

The most abundant soils in laterite profiles 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, 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.

The soils often have low densities but are overconsolidated and not highly compressible. The great depth of weathered materials can, however, result in significant settlements under the weight of a dam. The 37 m high Harris Dam in Western Australia (Somerford 1991, Bradbury 1990) is underlain by up to 30 m of extremely weathered granite forming the lower part of a lateritic weathering profile. Much of this material had dry densities of 1.35 t/m^3. The maximum foundation settlement on completion of construction was 460 mm and a further creep settlement occurred in the following two months.

Gordon & Smith (1984a, b) describe the results of field and laboratory tests on laterite soils developed on granites and dolerite near Worsely, Western Australia.

As indicated on Figure 3.42, *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' 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} 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.

Lateritic soils are usually non dispersive and, in the authors' experience, not highly erodible. Hence, the high permeability does not necessarily lead to unsafe dams, only dams which may leak significantly, if cutoff works are inadequate.

3.10.4 *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 Sasumua Dam in Papua New Guinea, 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.

Gidigasu (1976) presents a comprehensive account of the engineering properties and uses of lateritic materials, thoughout the world, including (Page 500) a table showing geotechnical properties of laterite soils used in the construction of embankment dams. Persons (1970) describes properties of lateritic soils in West Africa and Vietnam, and experience in the construction and maintenance of roads, airstrips and embankment dams.

3.10.5 *Lateritic soils – List of questions*

– Variable laterally and vertically?
– Deeply weathered?
– High insitu permeability, vertical and horizontal?
– Low insitu density?

– Fine soils suitable for earth core?

– Gravelly ferricrete or alcrete suitable for pavements?
– Cemented material in crust suitable for rockfill or rip-rap?

3.11 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 and Antarctica. 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 will therefore be limited to effects of the Pleistocene glaciation and of valley glaciers such as still occur commonly in alpine regions (Fig. 3.43).

Glaciated landscapes usually have complex histories of erosion and deposition, including for example, the following sequence, shown on Figure 3.44.

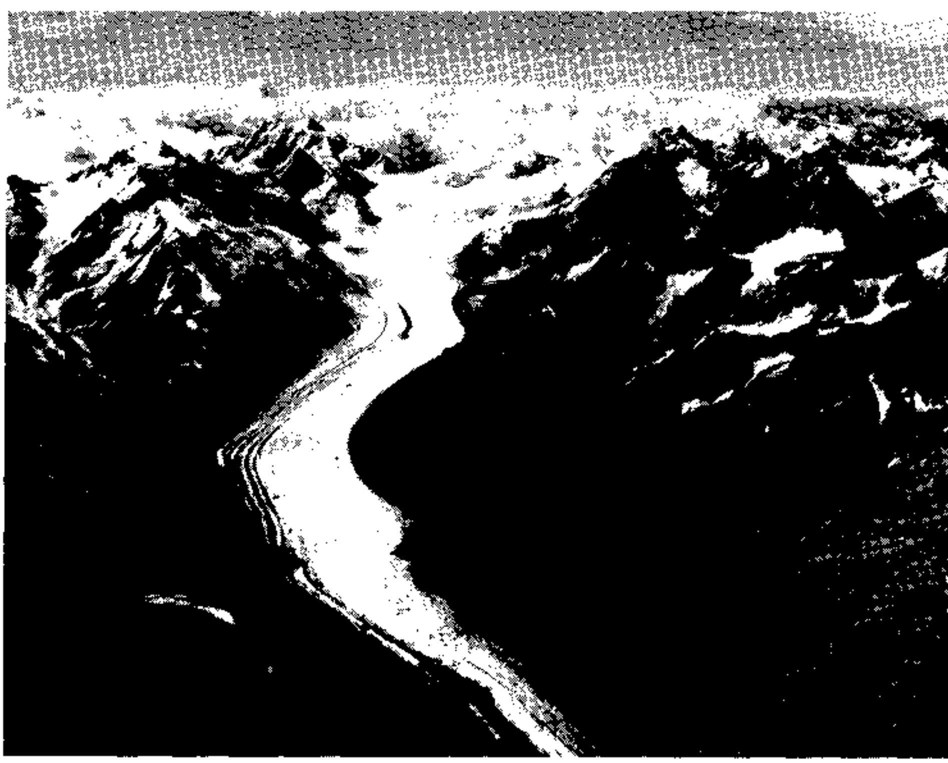


Figure 3.43. Aerial view of the Tasman Glacier in New Zealand. Photo courtesy of Mr Lloyd Homer, DSIR, New Zealand.

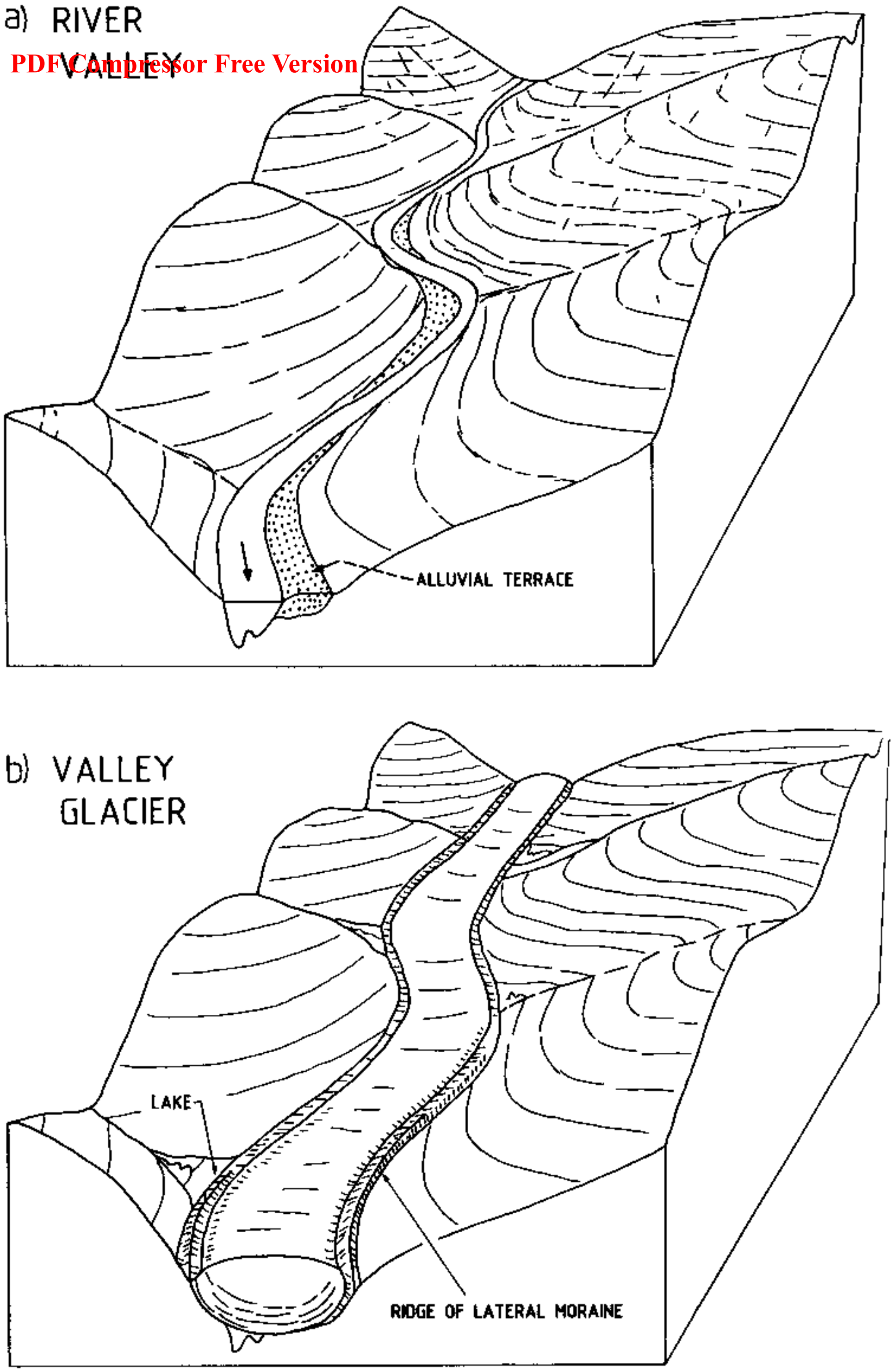


Figure 3.44. River valley, before during and after glaciation.

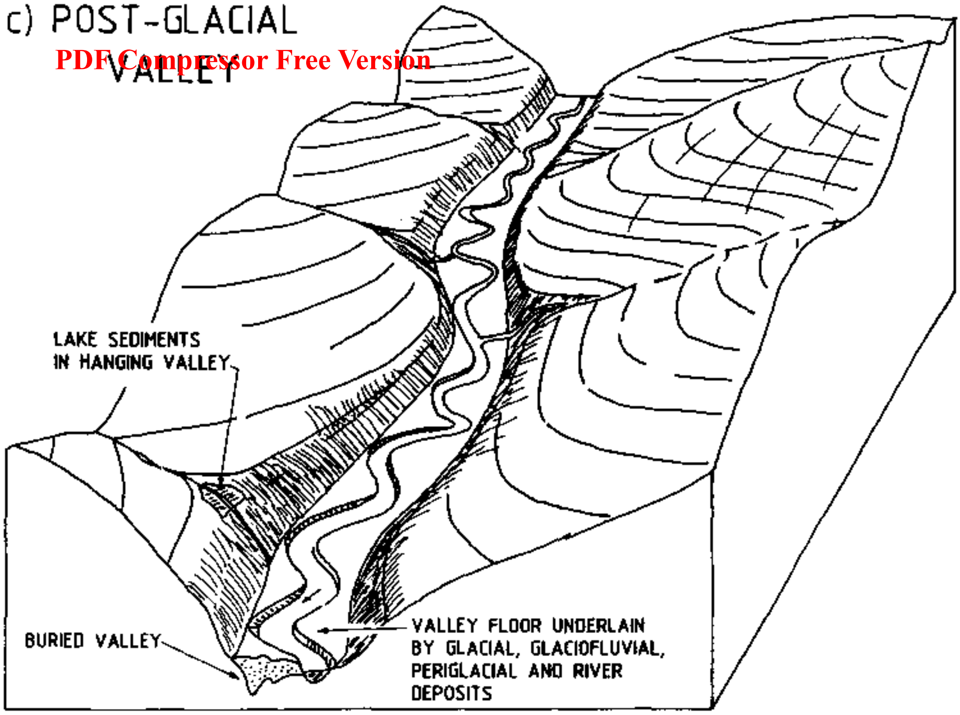


Figure 3.44 (continued).

– Erosion and shaping by rivers (Fig. 3.44a).

– Erosion and reshaping by ice, and deposition of glacially derived materials (Fig. 3.44b).

– Erosion and reshaping of the new landscape and the glacial deposits, by subsequent rivers (Fig. 3.44c)

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. However, 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 also about its likely distribution and engineering properties.

Boulton & Paul (1976) introduced the 'landsystem' form of terrain evaluation, as a means of classifying and mapping of sediment sequences and landforms. This landsystem approach is used by Eyles (1985) who recognizes three main landsystems resulting from glaciation.

– Subglacial } Characteristic of lowlands where sediments and land-forms were formed
– Supraglacial } by large ice 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.

– Glaciofluvial: In which glacially derived sediments were transported and deposited by water.

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

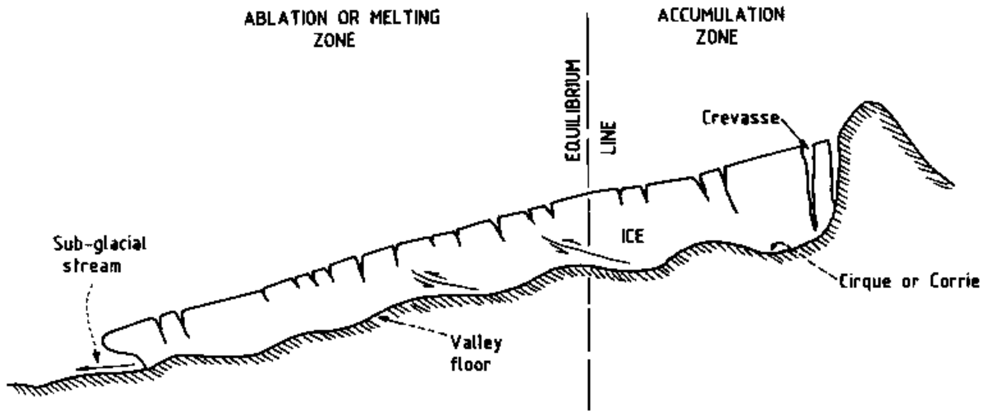


Figure 3.45. Diagrammatic section along a valley glacier (based on Blyth & de Freitas 1989; Fig. 3.38).

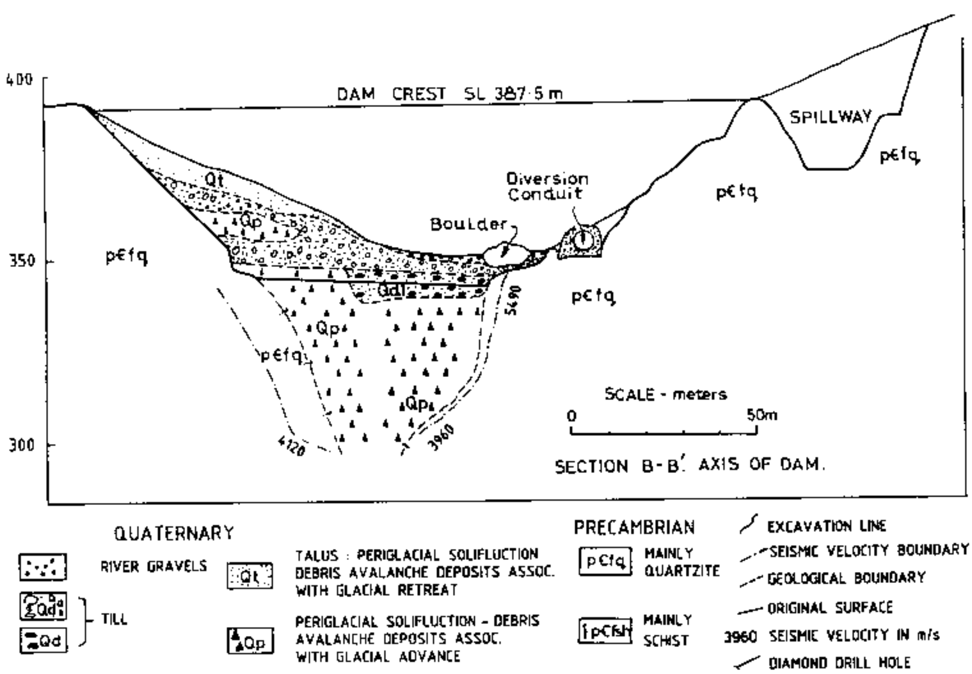


Figure 3.46. Parangana Dam, Tasmania, cross section along dam axis. (From Paterson 1971).

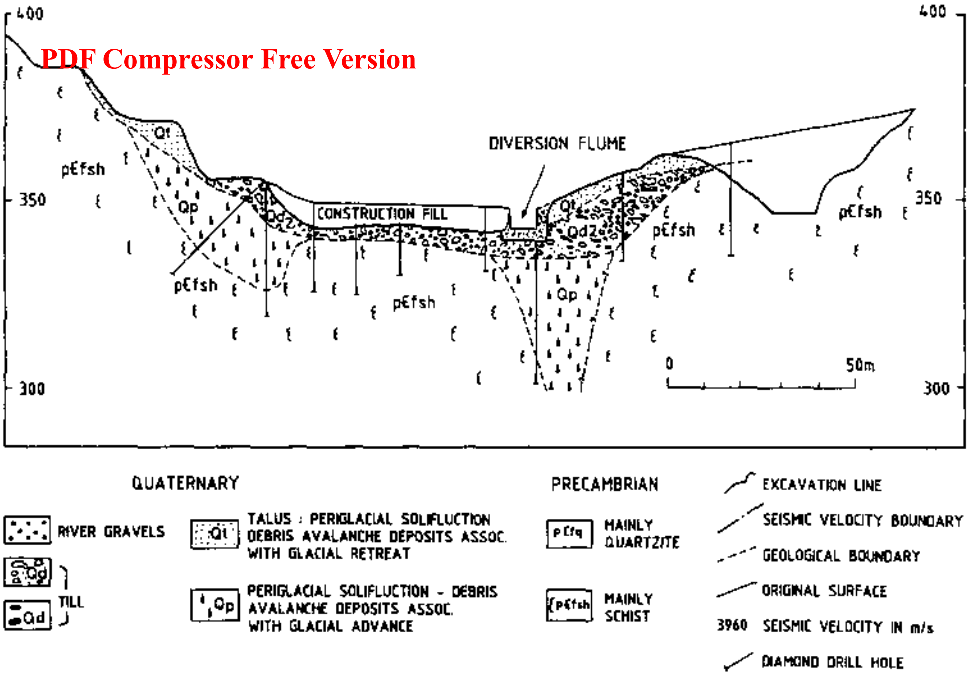


Figure 3.47. Parangana Dam, Tasmania, cross section 100 m downstream from dam axis (from Paterson 1971).

3.11.1 *Glaciated valleys*

Figure 3.45 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 being 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 (Figs 3.46 and 3.47). The shape of the buried valleys at this site and nature of materials filling them, 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.

3.11.2 *Materials deposited by glaciers*

The general 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.

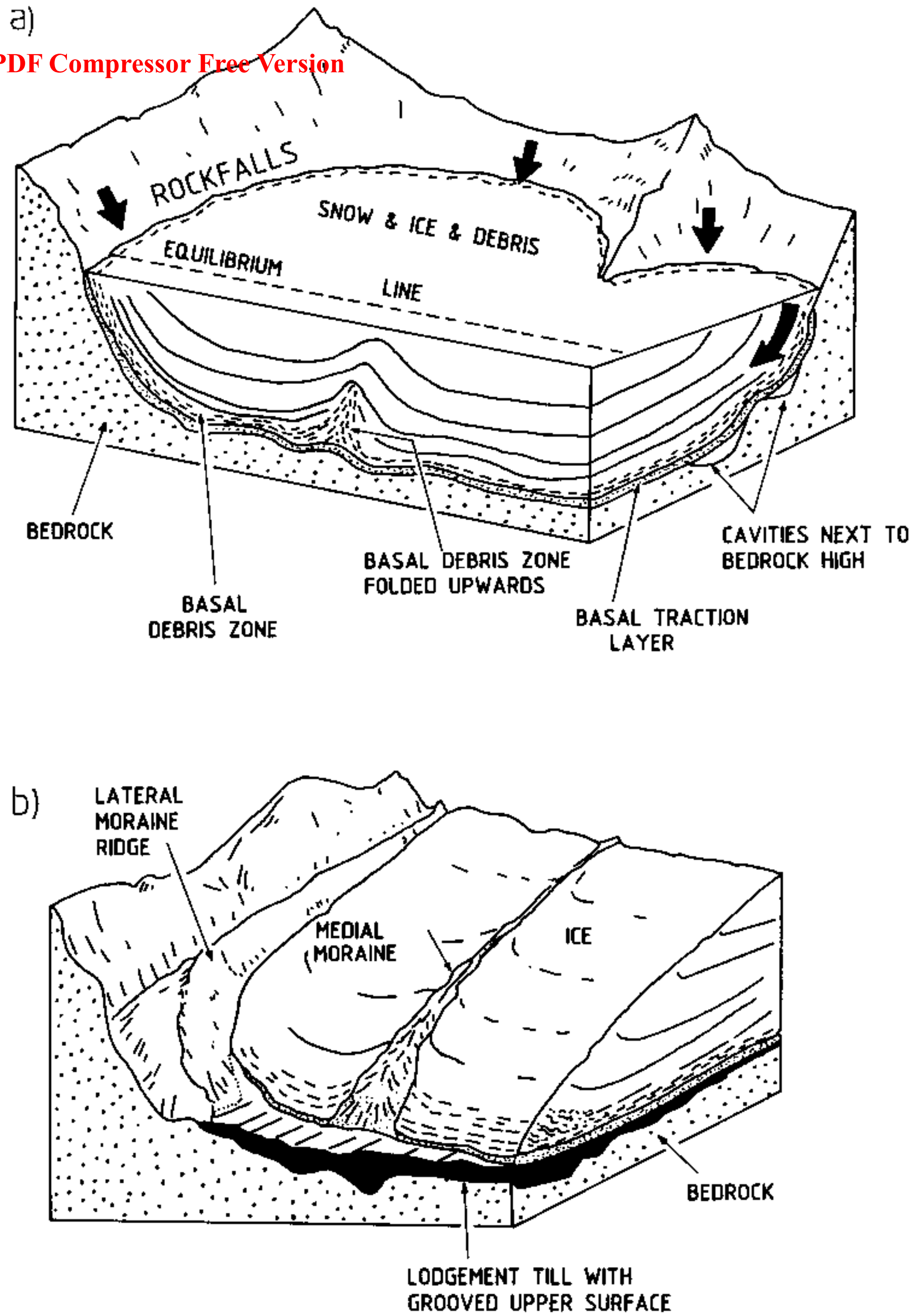


Figure 3.48. Debris transport by a valley glacier (from Eyles 1985, by permission of Pergamon Press).

Figure 3.48 shows mechanisms of accumulation, ingestion and transport of debris by a glacier. Upslope from the equilibrium line (Fig. 3.48a) 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 (Fig. 3.48b) 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.

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 (Fig. 3.48b) the debris may become exposed as a supraglacial medial moraine.

Below the equilibrium line (Fig. 3.48b) 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 3.49 shows a typical situation at the snout, where debris from the glacier surface and within it are deposited as the ice melts.

Figures 3.50 and 3.51 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 (Fig. 3.50a 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 and outwash materials are compressed and compacted by the glacier and their dips become steeper.

During the retreat (Fig. 3.50d) the glacier shrinks away leaving the lateral moraine as a ridge. Such ridges often remain steep (up to 70 degrees) 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 (Figs 3.50d and 3.51) by complex sequences including glaciofluvial sands and gravels (stream-deposits), laminated clays (lake-

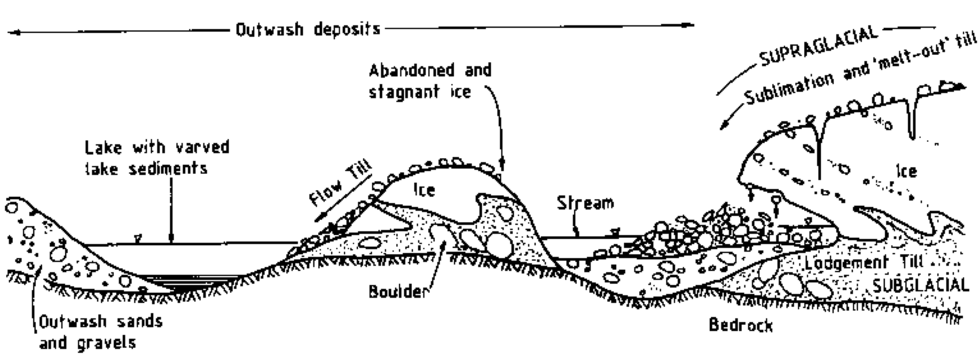


Figure 3.49. Diagrammatic longitudinal section at a glacier snout, and downstream (from Blyth & de Freitas 1989).

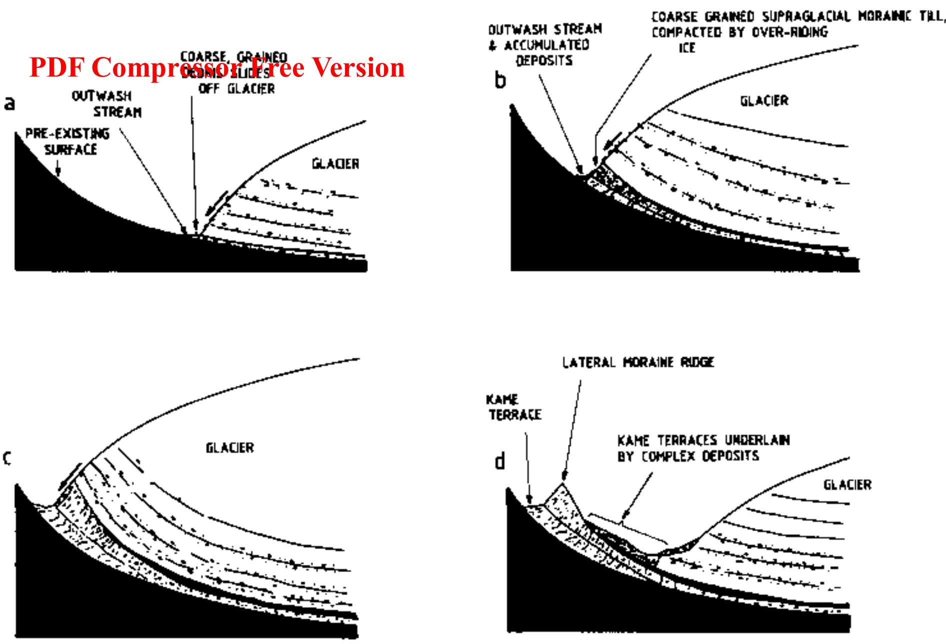


Figure 3.50. Development of a lateral moraine ridge and associated deposits (from Boulton & Eyles 1979).

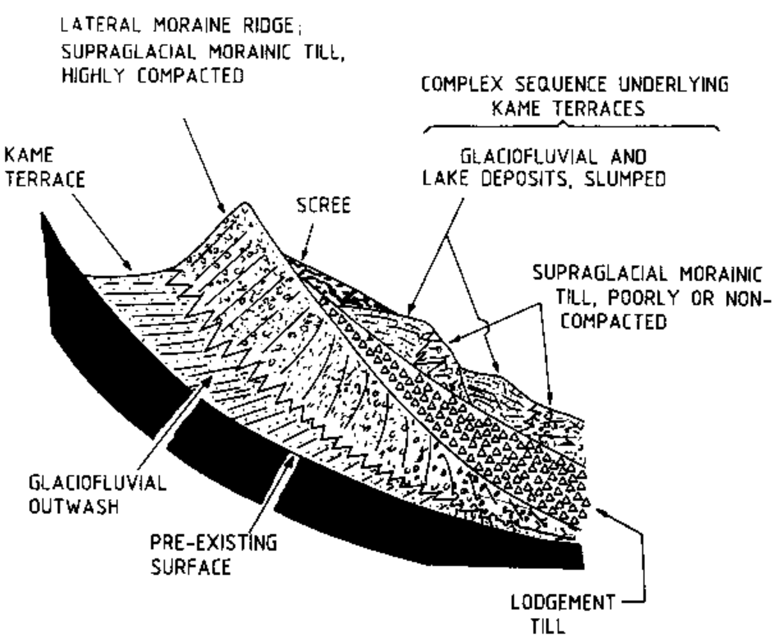


Figure 3.51. Diagrammatic section through a lateral moraine ridge (from Boulton & Eyles 1979).

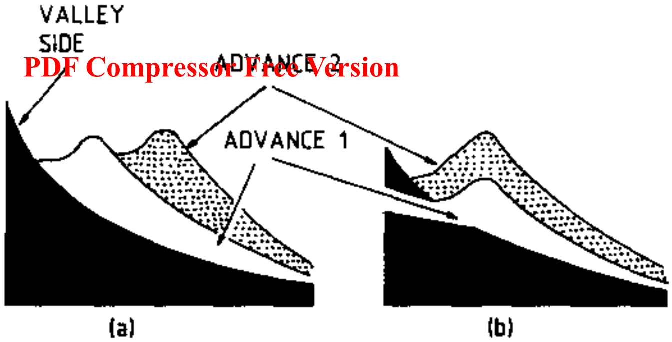


Figure 3.52. Lateral moraine ridges deposited by successive glacial advances (from Boulton & Eyles 1979).

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 (Fig. 3.51).

The lateral moraine and associated deposits are usually more complex than shown on Figure 3.51 because of repeated glacial advances and retreats. Figure 3.52 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 3.53 is the complete glaciated valley landsystem, 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 percent 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 by the overlying ice, has a well developed fissured fabric parallel to the ice flow direction, and is commonly highly deformed (Fig. 3.48). 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 (Fig. 3.47) 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, down 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 3.51.

Supraglacial morainic till (7) deposited at the snout as shown on Figure 3.49 shows 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 (Fig. 3.46) is believed to be of this type. It comprises up to 70 percent of gravel to boulder sizes, in a matrix of sand, clay and silt. The largest boulders are up to 3 m diameter.

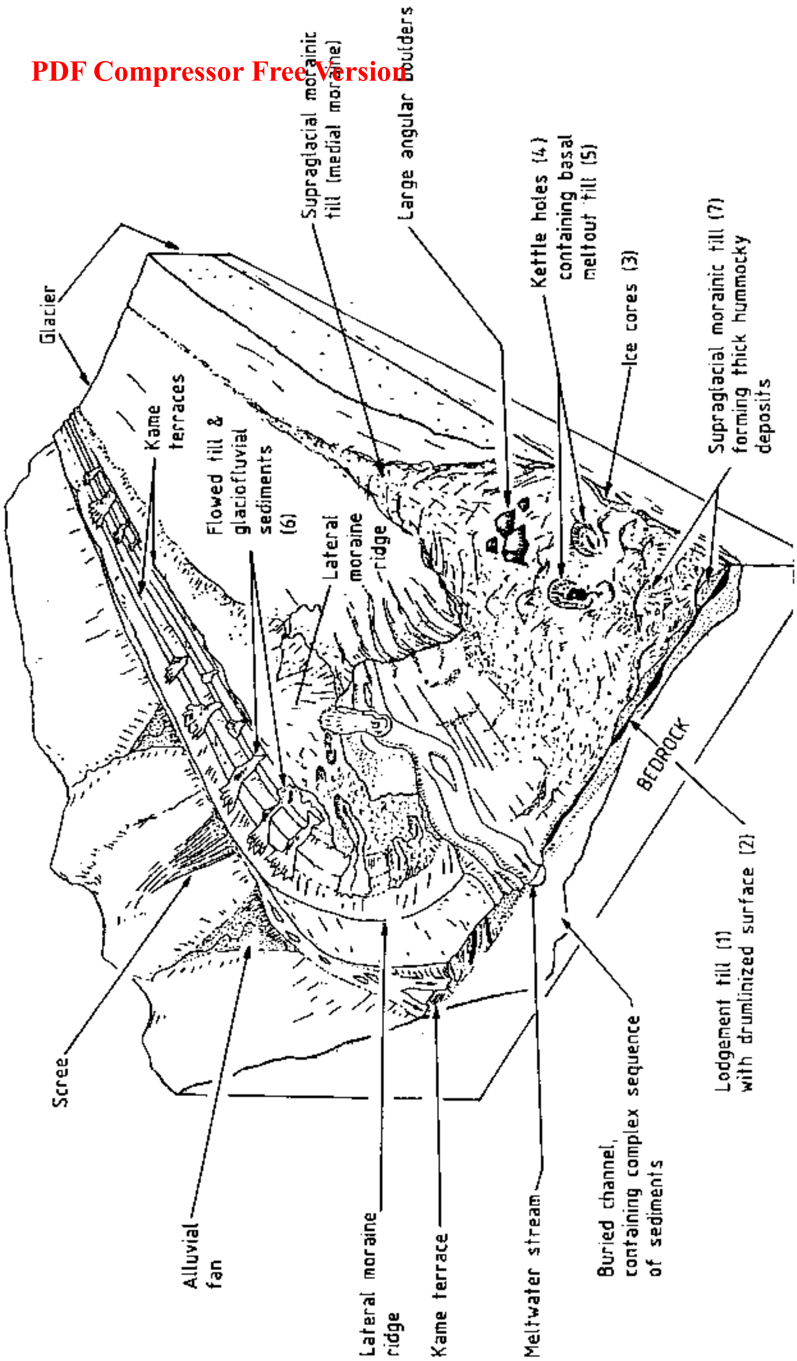


Figure 3.53. Landforms and sediments associated with the retreat of a valley glacier (based on Eyles 1985, by permission of Pergamon Press).

3.11.2.1 *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 it 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 3.50 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 (Fig. 3.46). The younger till was left in place beneath the downstream shoulder, but was covered with a filter blanket prior to placement of the rockfill.

Walberg et al. (1985) describe remedial works required at the 22.9 m high Smithfield Dam in Missouri, USA, after seepages and high piezometric pressures were recorded during first filling. New 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 & Wrigley (1985) describe further generalisations which can be made about the geotechnical properties of lodgement tills.

3.11.2.2 *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 2.1.3) 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 eg 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 rockhead 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.

1) 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?).

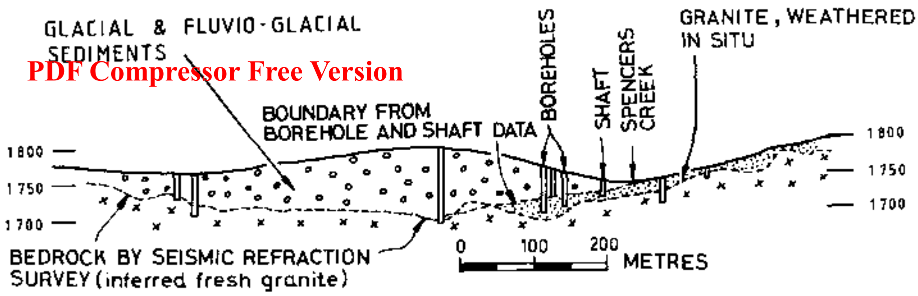


Figure 3.54. Cross section through the site for Kosciusko Dam.

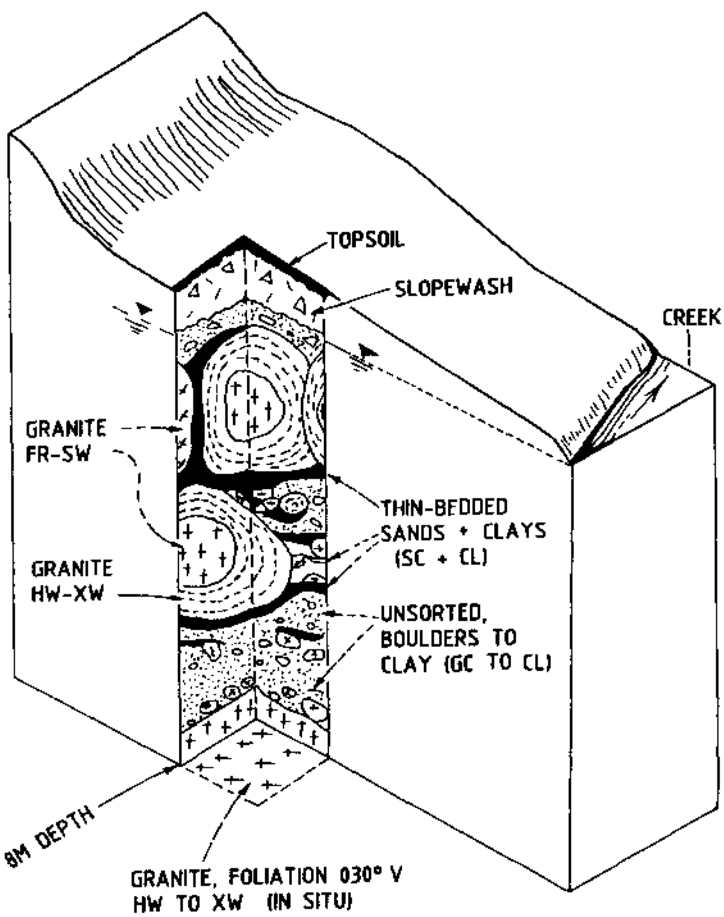


Figure 3.55. Log of shaft, Kosciusko Dam site.

2) 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.

3) 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 (Fig. 3.54).

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 to 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 (Fig. 3.55) 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 insitu weathered bouldery sequence. At 7.6 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 insitu.

Holes drilled elsewhere on the left bank recovered about 20 percent of fresh or slightly weathered granite in 'boulder' lengths, but little or no matrix. Hence the upper surface of the insitu 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 (Fig. 3.54).

3.11.3 *Glaciofluvial deposits*

As well as depositing some glaciofluvial materials in their immediate vicinity (as discussed in Section 3.11.2) glaciers release very large meltwater flows giving rise to deposition of vast amounts of gravels and sands in braided (multiple-channel) rivers usually extending tens of kilometres downstream, often across broad outwash plains (Fig. 3.56). Grainsizes 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.

The braided streams and lakes produce very complex, lenticular deposits of sands, gravels, silts and clays. Terzaghi & Leps (1958) describe the design, construction and performance of Vermilion Dam, a 38.8 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 north-western USA (see also Sections 3.8.1 and 3.8.2.2).

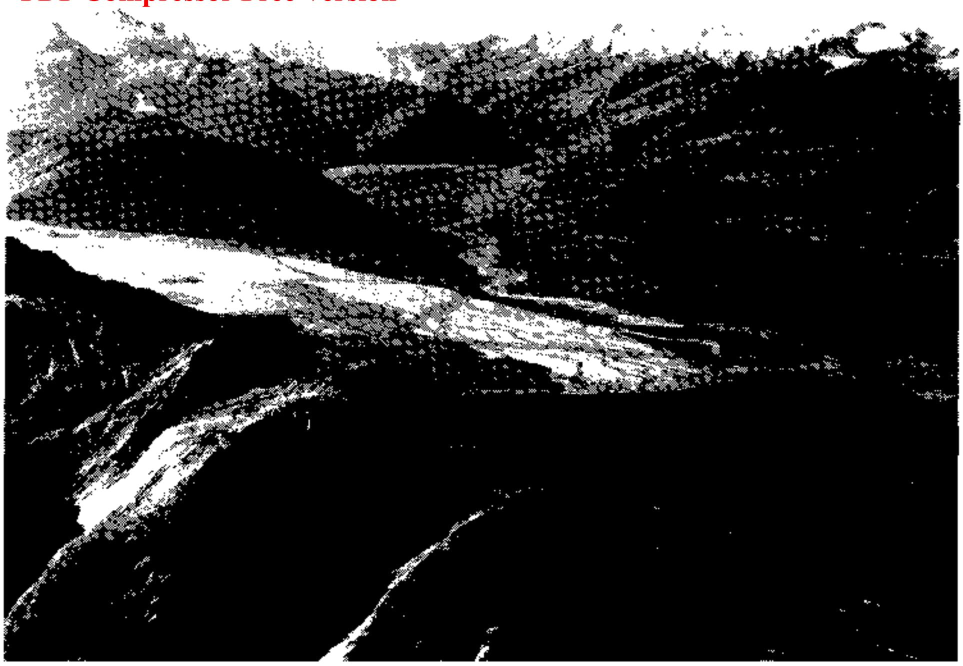


Figure 3.56. Broad valleys downstream from glaciers in New Zealand, underlain by great depths of glaciofluvial gravels and sands. Photo courtesy of Mr Lloyd Homer, DSIR, New Zealand.

3.11.4 *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 3.57, modified slightly from a diagram of Eyles & Paul (1985) shows the following features which may be developed under periglacial conditions.

1. Deep-seated creep into the valley, of weak sedimentary rocks. Competent beds develop widely gaping or infilled extension joints (gulls) and move downslope as complex slides or rafts on the weak materials.

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 – freeze-thaw and slopewash at the margins of snow patches.

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.

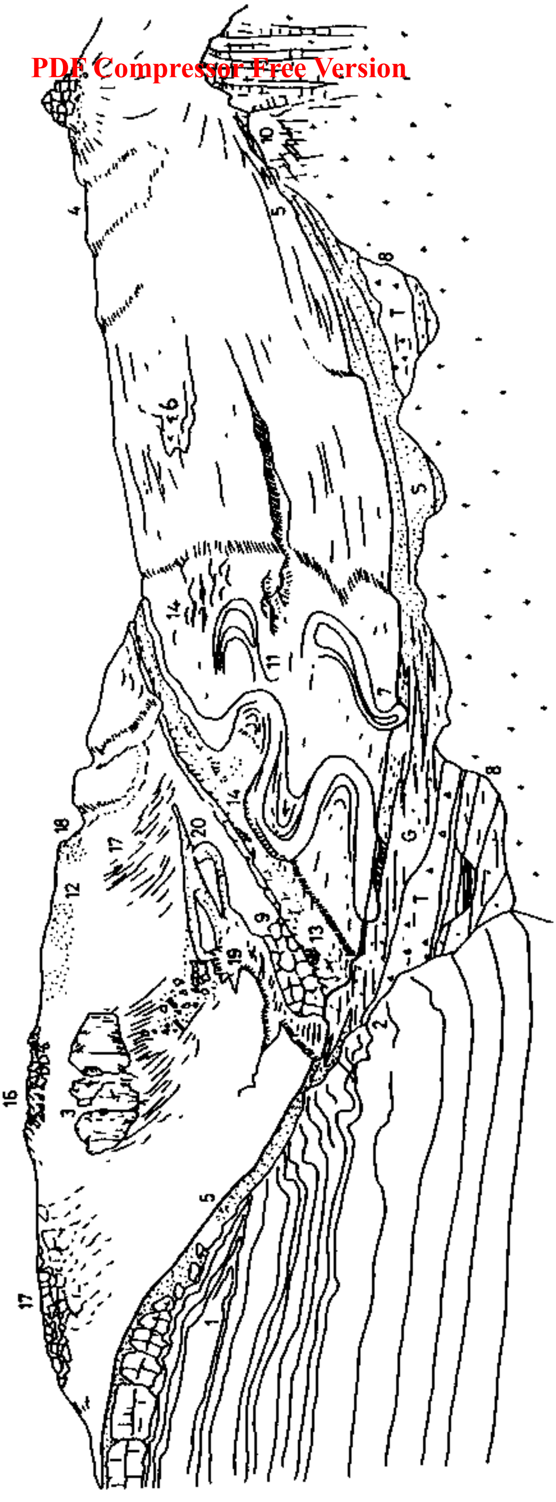


Figure 3.57. Diagram showing features which may be developed under periglacial conditions (based on Eyles & Paul 1985).

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 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 3.57 is based also on Eyles & 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 to 60 mm per year. Solifluction is caused by the generation of excess pore pressures during the thaw. Eyles & Paul (1985) provide a summary of theoretical studies of the process by Nixon & Morgenstern (1973) and McRoberts & 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 being 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 groundwaters have caused updoming of the surrounding sediments. Melting of the ice results in the doughnut-shaped surface features shown on Figure 3.57.

Paterson (1971) provides a useful account of periglacial features at Parangana Dam. Core-drilling into the steep sided buried pre-glacial river channels at the site (Figs 3.46 and 3.47) 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 (Figs 3.46 and 3.47). This material is believed to be a periglacial solifluction product formed during the final retreat stage of the glacier.

Soil and rock debris probably formed during the Pleistocene under periglacial conditions occurs widely in the southern highlands of New Zealand. In the Central Otago region where the bedrock is mainly fissile schist, this debris is commonly up to 50 m deep beneath valley slopes. Detailed studies for hydroelectric projects (Bell 1976, 1982, 1983; Gillon & Hancox 1992) have shown that many of these deposits are landslides, currently creeping at rates between 10 and 100 mm per year.

3.11.5 *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 3.53 and 3.57.

- 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?

CHAPTER 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 Chapter 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 cause 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 embankment; 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 countryside.

2. Foundation loading: Imposed by the embankment, raises questions of compressibility of the foundation and its stability against sliding upstream or downstream, before and after filling of the storage.

3. Inundation – Filling the storage : Causes changes to the groundwater regime, lowering of

Table 4.1. Geotechnical engineering questions.

1. Sources of materials, for the following purposes:
– Earthfill, impervious core.
– Filters.
– Rockfill.
– Rip-rap.
– Concrete aggregates.
– Pavements.

For each material: Location of alternative sources, quantities, methods for winning and processing, overburden and waste materials and quantities. Possible use of materials from required excavations, e.g. spillway or outlet works.

2. Reservoir
– Watertightness.
– Effect on regional groundwaters – Levels or quality.
– Stability of slopes inside and outside of reserovir rim.
– Erodibility of soils – Possibility of turbidity problems.
– Siltation rates and likely location of deposits.

3. Embankment
– Location – To suit topographic and geological situations.
– Alternative* sites, for comparison of costs and of geotechnical and other issues.
– Depths to suitable foundations for: earthfill; impervious core; filters; rockfill; plinth or grout cap.
– Nature of materials to be excavated, excavation methods, and possible uses of materials.
– Stability of excavations, support requirements.
– Permeability, compressibility and erodibility of foundations.
– Foundations treatment(s) required: grouting; slurry concrete; dental treatment; filter blanket; other.
– Embankment zones, methods of placement, methods of control of quality, moisture and compaction.
– Stability of embankment plus foundation in all situations.
– Monitoring systems: types, siting.

4. Spillway, river diversion works and permanent outlet works.
– Location and type.
– Excavation method(s), possible use for excavated materials.
– Stability of excavations, need for temporary/permanent support.
– Need for lining of channel or tunnel, protection of discharge area, or excavation of stilling basin.

5. Seismicity of region.
– Design earthquake.
– Maximum credible earthquake.

*Several alternative sites for the embankment are usually investigated during the feasibility stage.

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 embankment 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.

4. Flood discharges: Have high potential to erode, raising questions about the location and erodibility of discharge areas for the spillway and outlet works.

Table 4.1 is a suggested checklist of geotechnical engineering questions to be answered during site investigations for a typical embankment dam.

4.1.2 *Geological questions*

Typical geological questions for eleven common geological environments have been discussed and listed in Chapter 3. The following notes highlight the importance of asking and finding the answers to such questions, and of relating them to other site factors such as climate and topography.

4.1.2.1 *Questions relating to rock and soil types, climate and topography*

The relative importance of any one of the engineering questions on Table 4.1 and the amount and kind of site investigations needed to get the answer to it, will depend on the topographic, geological and climatic environments in which the project is to be located. For example, consider the effects of first filling of a water supply dam in an arid region, in a steep sided valley underlain by a very weak sandstone. The water table is likely to be very low or absent, and the sandstone may owe a large part of its strength and stiffness to cementation by water-soluble minerals such as gypsum or halite (common salt). Filling of the reservoir has the potential to cause dramatic changes to this site – significant raising of the water table, solution of water soluble minerals with resulting weakening and possible increase in permeability of the foundations, and possible instability in the storage area sides. Hence specific geological questions to be answered during investigations of this site would include the following:

– What are the cementing agents in the sandstone?

– How much reduction in strength and stiffness will occur in the sandstone when saturated for long periods?

– Could solution effects during dam operations result in increase in permeability of the foundation?

– Could solution/strength reduction or water table rise result in instability in (a) the foundation or (b) the reservoir sides?

At the other extreme, a site in a high rainfall area with gentle slopes underlain by very strong quartzite, and with a high water table, is likely to be almost unaffected by inundation.

It can be seen from these two very simple examples that certain generalizations can be made about geotechnical conditions likely to be met at a site, when its broad geological setting is known, and this is considered together with the site climate and topography.

4.1.2.2 *Questions relating to geological processes, i.e. to the history of development of the site*

It is not enough during the design and construction of a major dam, to know simply what rock or soil types are present, their engineering properties and their approximate distribution. Understanding the site geological environment implies also understanding the geological processes

Table 4.2. Processes which may be active enough to affect a dam project.

– Destressing	– Freezing
– Chemical weathering of rocks and/or soils	– Burrowing by animals
– Erosion by wind or water	– Growth of vegetation
– Depositions	– Rotting of roots of vegetation, or buried timber
– Creep, landsliding	– Seismicity, i.e. shaking, or displacement on a fault
– Subsidence	
– Pressure by groundwater	– Vulcanism

which developed the region and the site. The most important processes are usually the youngest, commonly those relating to the near-surface. This is partly because they will have had a major influence on the strength and stability of the valley slopes, and also because they may still be active, or may be reactivated by the construction or operation of the dam. Table 4.2 is a list of such processes, some of which have been discussed in more detail in Chapters 2 and 3.

4.2 GEOTECHNICAL INPUT AT VARIOUS STAGES OF PROJECT DEVELOPMENT

Most dam projects develop in five more or less distinct stages as set out on Table 4.3. Geotechnical input is required in all stages. That table summarizes the broad objectives of the geotechnical work and the usual activities and reporting needed, during each stage.

It is important during the planning stages (Stages 1 to 3) that the site investigation studies should proceed in phase with other engineering work being carried out at those times, eg. hydrological and topographic surveys, design and preparation of specifications. This is because most decisions affecting the project design or feasibility from the geotechnical viewpoint will affect the work requirements in the other fields. Conversely, decisions made from the results of these other studies will affect the scope of the geotechnical work.

It is desirable that the embankment designer takes a keen interest in the geotechnical work and that field and office consultations with the geotechnical team are frequent. Design drawings (e.g. cross-sections through structures) should include the main features of the foundation (i.e. the portion which Nature has made) as well as details of what Man proposes.

Timing and co-operation are just as critical during the construction stage. During this stage the foundation rock or soil is exposed better than ever before and (hopefully) after, and if the site investigator arrives too late, the exposed area showing a critical clue can be covered by the first layer of fill. It goes without saying that unless the site investigator has the confidence and co-operation of the construction team, this phase of the work will not be as fruitful as it otherwise could be. In extreme cases, lack of a competent geotechnical observer and advisor during the construction stage can prove disastrous – resulting in either expensive contractural disputes or later failure of the structure.

Similarly, during the operation stage inspections must be made at regular intervals, to ensure that any malfunctions are discovered while there is still time to remedy them. Such inspections are carried out as part of a surveillance programme as discussed in Chapter 18.

4.3 AN ITERATIVE APPROACH TO THE INVESTIGATIONS

During the planning stages the geotechnical studies should be carried out using the iterative approach shown on Figure 4.1 which is based on ISRM (1975) and Stapledon (1983). The following notes relate to the seven activities on Figure 4.1.

Activity 1. First, the objectives of the work, or questions to be answered, are defined. As discussed in Section 4.1, these will include both engineering and geological questions. The number of geological questions defined at this time might be quite small, if little is known of the geology of the region or site.

Activity 2. Existing geological and other data relevant to the site are collected and compiled to give a tentative 'geotechnical model.' Tentative answers to the questions asked in Activity 1

Table 4.3 Geotechnical input during stages of development of dam project.

Stage No.	Name	Objectives and activities (Geotechnical)
1	Pre-feasibiliy	Assist in the selection of possible sites and obtain enough understanding of the geological situation to plan the feasibility and site selection studies. Usually includes review of existing data plus a rapid air and ground inspection
2	Feasibility and site selection	Assess the project feasibility from the geotechnical viewpoint considering both the regional and local geological situations. Explore alternative sites for dam and other key structures, and adopt the most promising sites. Explore these further if necessary to confirm feasibility and provide sufficient data for preliminary design and feasibility stage cost estimate. Provide regular progress reports and prepare formal report at the end of Stage 2 with a definite statement confirming (or otherwise) the project feasibility from the geological point of view
3	Design and specification	Answer any questions outstanding or arising from the feasibility studies, and additional geotechnical questions raised during the design. Further site exploration and testing usually necessary. Provide regular progress reports, and report for tenderers or construction agency. Provide assistance in preparation of the specification
4	Construction	Ensure that the geological picture exposed during construction is as assumed in the design, and if not, that modifications are made to the design, if necessary. Provide day to day advice on geotechnical matters, to the resident engineer. Provide record of geological exposures during construction, and of any rock movements, water inflows, etc. in regular progress reports. This data is vital to the Surveillance Group (see Stage 5); should any malfunction develop it may form the main basis for (sometimes rapid) corrective action. Activities include detailed mapping, colour photography, review of inspector's records, installation of instruments, simple tell-tales, etc.
5	Operation	Ensure that the structure is performing as designed, from the geotechnical point of view, and assist in the design of remedial measures, if it isn't. Inspect the completed structure, the site area, and records from instruments and operator's observations, at regular intervals, preferably as a member of a surveillance committee, including representatives of design, construction and operation branches. Should any malfunction be evident, assist in design of remedial measures. This could involve conducting further site investigation and/or analysis of the geotechnical records from Stages 1 to 5

are obtained where possible from either local knowledge or from rapid analyses (see arrow to Activity 7) or both. New geological questions are usually added at this stage, arising from the understanding obtained from the existing data.

Activities 3 and 4. To answer the remaining questions and to confirm the tentative answers, the various sub-objectives and activities of Activity 5 to 7 are defined. Some of the activities are essentially geological and others essentially engineering in nature. The activities are related to time and money respectively, in activity charts and cost estimates, and set out in a report to management, seeking approval to proceed.

Activity 5. This relates to achieving all of the esssentially geological sub-objectives defined in Activity 3. The 'engineering geological model' here implies a sufficient understanding of the

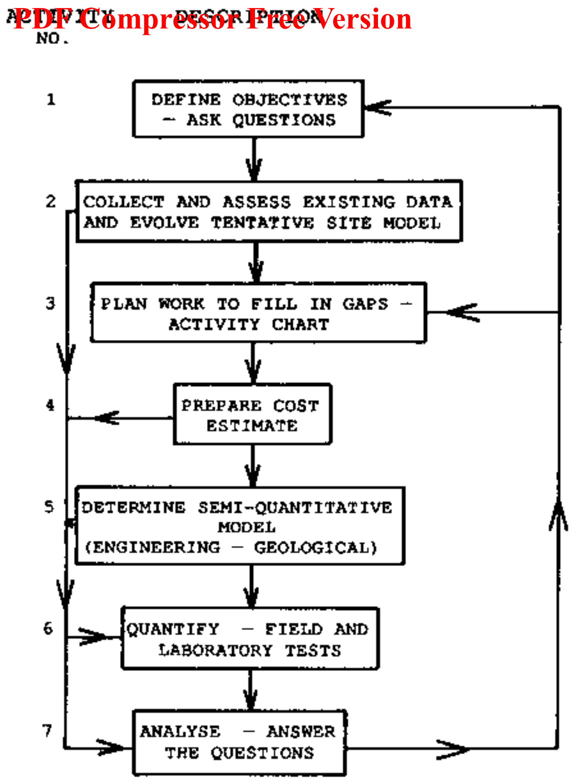


Figure 4.1. Activity flow in site investigations

regional geology, geological history, and detailed site geology. The model is described in geotechnical language.

Activity 6. Field or laboratory tests, or both, are carried out to obtain quantitative values – usually for engineering properties of critical portions of the detailed site model. The test results are assessed in the light of the geological model and from this, certain values or ranges of values may be adopted as realistic, and included in the model which is now termed the geotechnical model.

Activity 7. Engineering analyses are then carried out involving the proposed structure and the site geotechnical model. For the parts of the model without test results, assumptions are made based on precedents and knowledge of the likely variability of the types of materials present. It is desirable that the analyses provide answers in terms of probability of failure, as well as factor of safety.

Usually many questions will have to be answered by judgement, based on experience in similar geological situations.

If at the end of Activity 7, all questions are answered with sufficient confidence, the investigation is complete. If not, further cycles of investigation are carried out until the required level of confidence is reached. The 'law of diminishing returns' is applied.

4.4 PROGRESSION FROM REGIONAL TO LOCAL STUDIES

The geological studies under Activity 5 start with consideration of the site location in relation to the global tectonic situation, and should include study of the geology of a broad region surrounding the site. This is necessary to assess the effects on the project of large scale processes, some of which (Table 4.2) may have potential to damage it.

The regional geological studies are followed by studies at and near the site on intermediate and detailed scales. The objectives and usual activities of the regional and detailed studies are set out below, generally following ISRM (1975). More detailed descriptions and discussion of the various activities are presented in Chapter 5.

4.4.1 *Broad regional studies*

4.4.1.1 *Objectives*

1. To provide an understanding of the geological history of the project area, that is, of the processes which have developed the present geological situation at and in a broad region around the site. In particular it is important to determine the following:

– any processes that are active or potentially active, e.g. as listed on Table 4.2;

– the possible effects of any active processes on the proposed works both during construction and in service;

– whether any construction activities (e.g. excavation) or operation of the storage are likely to cause such changes to the existing regime (e.g. stress or hydrologic) as to require remedial works or to affect the project feasibility.

2. To determine the regional stratigraphy and geological structure.

3. To explain the geomorphology of the project area in terms of the regional stratigraphy, structure and geological history.

4. To draw attention to important features, eg. major faults or landslides, occurring at or close to the site, but not exposed or recognisable at the site.

5. To get an appreciation of the regional groundwater conditions.

6. To form a logical basis for the location and proving of sources of construction materials.

4.4.1.2 *Activities*

The amount and money spent on regional studies will depend upon the size and complexity of the project, its hazard rating, and the amount and relevance of regional information already available. The activities usually will include some or all of the following:

1. Examination of existing regional geological maps, cross sections and reports.

2. Interpretation of satellite images and aerial photographs.

3. Interpretation of ground photographs.

4. Ground reconnaissance over previously mapped areas, and remapping of important areas with the project objectives in mind.

5. Compilation of regional plans and cross sections showing the proposed project works.

4.4.2 *Studies at intermediate and detailed scales*

4.4.2.1 *Objectives*

1. To explain the development of the site topography in terms of the regional and local geology and geological history.

2. To delineate any major features, e.g. folds, faults or landslides, which may or may not be indicated already by the regional studies.

3. To provide an 'engineering geological model' showing the nature and distribution of rock and soil substances and defects at and close to the site.

4.4.2.2 *Activities*

1. Detailed geotechnical mapping.
2. Studies of the pattern of defects using the stereographic projection method.
3. Application of geophysical methods (commonly seismic refraction) and interpretation of the results in the light of the geological picture.
4. Planning and technical supervision of direct exploration, which can include core (or other) drilling and excavation of trenches, pits, shafts or tunnels.
5. Logging of the cores of drill holes and the exposures in the exploratory excavations, using geotechnical terms.
6. Logging of the sides of drill holes by means of periscopes, cameras or impression devices.
7. Plotting of all geological and geophysical data, as soon as it is obtained, onto plans, sections and three-dimensional models (either physical or computer generated). In this way it is possible to analyse the surface and subsurface data as they become available, progressively filling significant gaps in the geological picture until the required degree of detail is reached. All plans, sections and models show the proposed works at least in outline.

4.5 REPORTING

It is important during all stages, that the geotechnical facts, interpretations, conclusions and decisions made from them are recorded regularly by a system of formal progress reports. A comprehensive report is essential at the end of each stage, setting out the answers to the questions of that stage, and with recommendations for the next stage.

In general terms, a geotechnical report should consist mainly of a carefully planned and ordered sequence of drawings (and preferably some photographs) with relatively short explanatory text, and appended tables and calculations. The drawings and photographs should convey a clear three-dimensional picture of the surface and subsurface conditions at the site. A clear distinction should be made between factual data and inferences made from them.

A formal system for checking and certification is needed for all drawings and reports.

4.6 TIMING AND FUNDING OF GEOTECHNICAL STUDIES

It has to be appreciated by dam-building organizations that geotechnical site studies usually cannot be planned, time-scheduled or costed with anywhere near the same precision as construction work. At a site with deep soil cover and no existing local or regional geological maps there will initially be no clear indication of what might be found, and so planning of a major site exploration programme would be foolish. The exploratory work at such a site would best proceed in short stages, with the activities in each successive stage being designed to answer questions arising from the preceding stage.

In the opinion of the authors, it is most important that dam projects receive adequate funding at the feasibility stage. This is because the question 'Is it economically feasible to build and maintain a safe structure at this site?' is the really vital and difficult one. Usually there are alternative sites but finally one must be adopted and the above question must be answered in relation to it.

Unfortunately, some organizations may not have (or seek?) large amounts of money until they believe they have shown a project to be feasible. Also, they may not wish to, or cannot, acquire the site until feasibility is assured. Because of these matters, and the inevitable pressure from landowners and conservation groups, the investigations at this vital stage often tend to be less logical and less thorough than necessary – too much reliance is placed on indirect methods (geophysical) and drilling of small diameter boreholes. The advantage of these methods is that they cause minimal disturbance of the land. However, without an adequate understanding of the geological situation, which often can be obtained only by large, continuous exposures in deep bulldozer trenches or access track cuttings, the results of geophysical studies and drilling can be difficult to interpret. For major, high hazard dams at geologically complex sites, exploratory adits are often the most appropriate method for answering the feasibility questions.

The consequences of a poorly planned or inadequately funded feasibility stage investigation can be:

– adoption of the less satisfactory of two alternative sites.

– adoption of a site which later proves to be non-feasible (for examples of such sites see Table 3.4).

– abandonment of a site (or possibly a whole project) which would have been proven feasible, by well conceived (but expensive) studies. The authors consider that this might have been the fate of Thomson Project, if the downstream ridge stability question had been raised during the feasibility stage (see Chapter 2, Section 2.6.3.5).

The authors consider that the 'observational method' as adopted by Terzaghi & Leps (1958) at Vermilion Dam, should not be used as a means of overcoming funding difficulties during the feasibility and design stages of dam projects. They agree with the practice of excavating part or all of the dam foundations during the design stage, or prior to awarding the main construction contract, but believe that this should be done only when all questions relating to feasibility of the project have been adequately answered.

4.7 THE SITE INVESTIGATION TEAM

The following seven attributes for success in site investigations for dams, put forward by Stapledon (1983), suggest that a team approach is necessary, especially for large dam projects:

1. Knowledge of precedents.
2. Knowledge of geology.
3. Knowledge of soil and rock mechanics.
4. Knowledge of civil engineering design.
5. Knowledge of civil engineering construction.
6. Knowledge of direct and indirect exploratory methods.
7. Above average application.

It should be clear from Section 4.1 to 4.4, that Attributes 1 to 5 are necessary for understanding, defining and answering the geotechnical questions associated with embankment dams.

Attribute 6 refers to direct or subsurface exploratory methods (e.g. pits, boreholes) and indirect methods (e.g. geophysical transversing). Knowledge of the application and limitations of methods such as these is necessary if the questions posed are to be answered effectively and economically.

The authors consider that these six attributes are necessary for successful site investigations for dams. They also consider that these attributes can be provided most effectively by a team including engineering geologists and engineers. It is important that all team members have enough 'general' knowledge in all areas to be able to communicate effectively with one another and with engineers involved with the design, construction and operation of dams. As well as having this broad knowledge of the industry, it is desirable that they have Attribute 7, 'above average application' as defined by Stapledon (1983) '...ability to get things done; this involves effective cooperation with others, enthusiasm and drive and the ability to make decisions in the field.'

The authors have asked themselves whether it would be satisfactory for both the geological and engineering inputs required could be provided by a team of 'geological engineers.' They consider that usually this would probably not be satisfactory for large dam projects, for the following reasons:

1. The leader of the team usually should be the designer of the embankment – a geotechnical engineer who has specialized in such design work, rather than geological aspects.

2. Site investigations for dams require higher levels of understanding of geological processes and regional geological assessment (in particular) than that needed for site investigations for most other structures. Such levels of understanding in these areas are likely to be achieved only by persons with thorough basic training and experience in them.

3. Experience has shown that very few persons including those who hold degrees in both engineering and geology, have been able to perform satisfactorily in the roles of both engineer and engineering geologist.

CHAPTER 5

Site investigation techniques

Several techniques or 'tools' may be used in a dam site investigation designed to follow the broad framework explained in Chapter 4. This chapter discusses some of the most common techniques, their applicability and limitations. It is emphasised that use of several techniques is always required, as restriction to a single investigation method would be unlikely to yield correct answers to the site questions in an economical manner.

5.1 TOPOGRAPHIC MAPPING AND SURVEY

A fundamental requirement for the investigation and design of any project is accurate location and level of all relevant data. Topographic maps at suitable scales are essential with establishment on site of clearly identified bench marks. All features recorded during the investigation should be located and levelled, preferably in relation to a regional coordinate system and datum. A local system may be established provided that at some stage during the investigation the relation between the regional and local survey systems is determined.

These statements may appear obvious, but in many instances interpretation, construction and related contractual problems have been shown to have originated from poor survey control. Topographic maps at several scales are required. The actual scale will depend on the size and complexity of the area but the following common scales are given as a guide:

– Regional maps, 1:250 000 with 20 to 40 m contours to 1:25 000 with 10 m contours.
– Catchment area, 1:25 000 with 10 m contours to 1:2000 with 2 m contours.
– Project area, 1:1000 with 2 m contours to 1:250 with 1 m contours.
– Individual engineering structures, 1:500 with 1 m contours to 1:200 with 0.5 m contours.

Regional topographic maps issued by government agencies in most countries are published at several standard scales which range from 1:250 000 with contours at 20 to 40 m to 1:25 000 with contours at 10 m interval. The user of these maps should consider their original purpose and the accuracy of the information plotted on them in relation to the project requirements. The notes on the map which indicate the method of compilation – whether aerial photogrammetry or ground survey – and the date of preparation of the map, may be relevant.

Photogrammetry can be used in the preparation of project specific plans provided the aerial photographs have been flown at low level and accurate ground control points are identified on the photographs.

Photogrammetry is often not adequate in steep, tree-covered areas. The authors are aware of two projects where errors of up to 20 m in elevation have occurred due to inadequate allowance for tree cover. This resulted in the requirement for substantially larger saddle dams than estimated in the feasibility studies and significantly affected the viability of the projects.

The field survey team should liaise with the site investigation group, and provide a series of clearly labelled ground control marks throughout the project area to assist in the location of features identified during the geotechnical mapping. All boreholes, pits and trenches should be located, levelled and clearly labelled.

5.2 INTERPRETATION OF SATELLITE IMAGES AND AERIAL PHOTOGRAPHS

Satellite image maps and aerial photographs are available from government agencies in most countries.

5.2.1 *Interpretation of satellite images*

Standard LANDSAT images are at 1:1 000 000 scale but enlargements at 1:500 000 and 1:250 000 are also available.

These small scale images or image maps provide a broad view of the region in which the project is to be located. This broad view can indicate correlations between geological features, or the position of geological boundaries or faults, when these features are of such great extent that they are not recognisable on larger scale photographs covering smaller areas (Norman 1980).

The broad view also provides an indication of relationships between the regional geology and landforms, drainage, soils, vegetation and land-use, which may be useful in:

– planning of access routes to and within the project area,
– location of potential sources of construction materials, and
– assessing reservoir siltation rates.

5.2.2 *Interpretation of aerial photographs*

5.2.2.1 *Coverage*

If the existing aerial photographs in the project area are inadequate in quality, coverage or scale, it is best to take new photographs with the following advantages:

– the photographs will show present conditions,
– the required scales and coverage can be specified, and
– the photographs can be used to prepare topographic maps.

5.2.2.2 *Interpretation*

Photogeological interpretation using a stereoscope forms a major part of the initial appraisal of regional and local site conditions during the pre-feasibility and feasibility stages (see Section 4.4.1.2 and Table 4.3). It usually gives a good indication of likely geotechnical constraints on the project and the extent of investigations required.

Much has been written about the techniques of photogeological interpretation and their application in engineering (Rib & Liang 1978, Rengers & Soeters 1980, Bell 1983, Lillesand 1987, and Hunt 1984). Comments will therefore be limited to aspects of special significance in dam engineering.

Figure 5.1. Aerial photograph showing folded sedimentary rocks in Central Australia.

The stereoscope provides a three-dimensional image with a vertical scale which is exaggerated by a factor which depends on the distance between the photo-centres. This exaggeration can be of benefit as it highlights surface features which are actually more subtle, but interpreters should realise that the ground slopes viewed under the stereoscope appear steeper than they really are.

A major advantage of aerial photographs is that distance of observation is not impeded by relief. It is possible to observe features on both sides of a hill at the same time and thus establish their continuity, if it is present. This is particularly important in the interpretation of geological structure.

Landforms which reflect the structure of folded rocks show up well on aerial photographs.

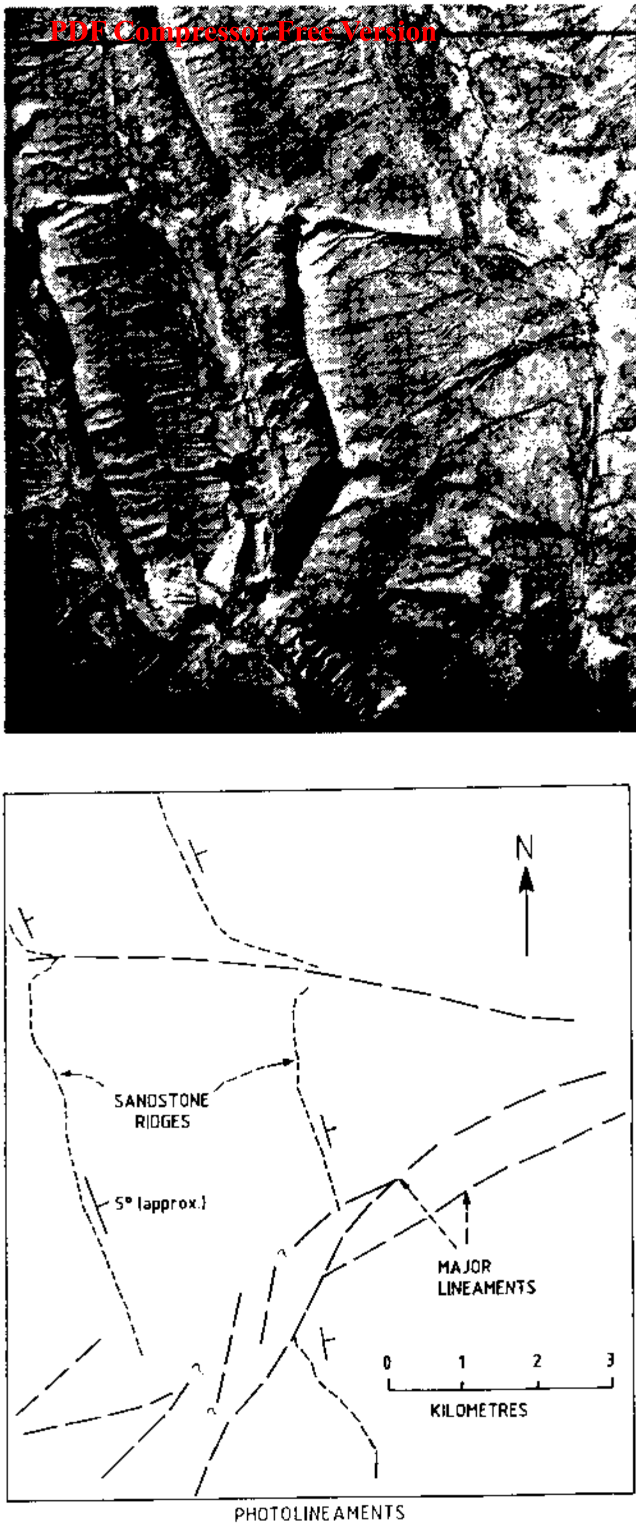
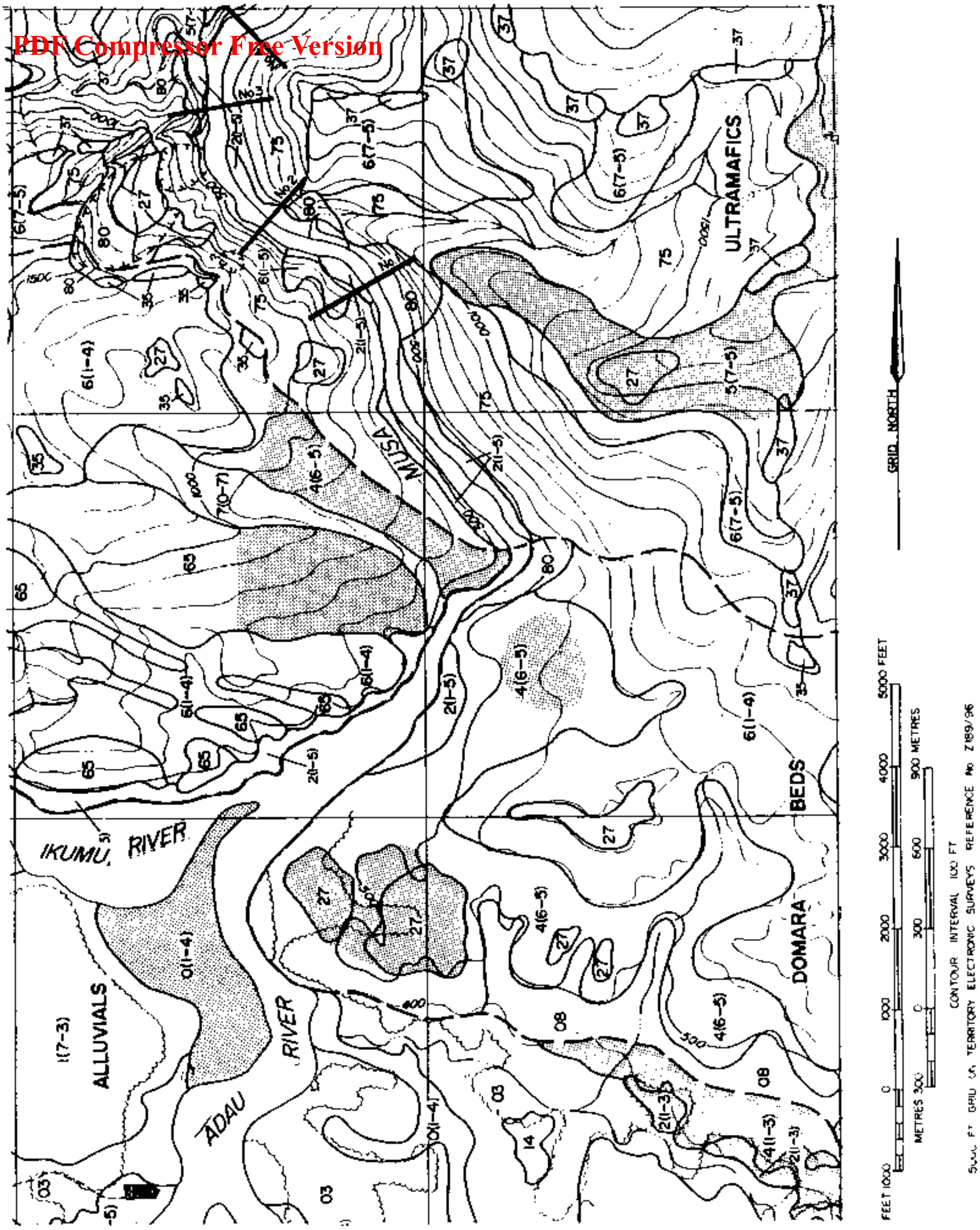


Figure 5.2. Aerial photograph and sketch showing lineaments in an area underlain by gently dipping sandstones.


ULTRAMAFICS
MUSA
IKUMU RIVER
ALLUVIALS
ADAU RIVER
DOMARA BEDS
GRID NORTH
FEET
METRES
CONTOUR INTERVAL 100 FT

Village

Track

River, stream

Treeline

1500 Contour in feet (amsl.)

RIVER Control point TES ZI89/96

No.3 Dam site

Geological boundary

Terrain boundary

Landslide area

Potential borrow area (subject to investigation)

NOTE. The base map is compiled from CDW Drawing Nos. PH68/70 & PH67/225. The reliability of the map position on the grid, north of 335,000N, is fair.

TERRAIN CLASSIFICATION FRAMEWORK & EXPLANATORY NOTES

GEOLOGY		TERRAIN MAPPING UNITS						
GEOLOGICAL GROUP	LITHOLOGY		No.	PHYSIOGRAPHIC FEATURE	NATURAL SLOPE RANGE	No.	SOILS	U.S.C. SYMBOL
ULTRAMAFICS: [(Kgd) & (Kpd) undifferentiated]	Massive noncumulus peridotite, dunite – commonly layered and in places brecciated; minor gabbro.	Horizontal or Sub horizontal Surfaces	0	Level or depressional, poorly drained areas.	Level or very low gradient.	0	Rock outcrop or skeletal soils.	–
			1	Level, very gently undulating or very gently sloping surfaces – well drained areas.	Level or up, to 2°.	1	Gravels – (alluvial or colluvial).	GC-GM-GW
DOMARA BEDS: [Includes Domara River Beds (Qpd) and minor Musa Volcanics (Qpm)]	Conglomerate, siltstone, shale with minor intermediate to basic agglomerate, tuff and basic lava.		2	Undulating or irregular surfaces.	Variable – predominantly long, gentle slopes (to 8°).	2	Sand – Uniform profile.	SW-SM-
			3	Strongly undulating, irregular or hummocky surfaces.	Variable – predominantly short, moderately steep slopes (8° – 18°).	3	Sandy or silty medium textured soils (gradational, uniform or duplex profile).	SM-ML or SC-CL
ALLUVIALS: (Oa)	Unconsolidated gravel, sand, silt and clay, local areas of boulder alluvium.	Sloping Surfaces	4	Slopes gentle (hill slopes; foot slopes etc.)	to 8°	4	Silty soils or stratified soils – predominantly silty.	SM-ML-MH
			5	Slopes moderate. (hill slopes, escarpment slopes etc.)	8° – 18°	5	Gravelly or stony clay, silty clay, sandy clay etc.	GC-CH & MH*
			6	Slope steep (hill slopes, escarpment slopes etc.)	18° – 30°	6	Duplex soils – medium to coarse textured soil over fine textured soil.	SP-SC/CL-CH*
			7	Slopes very steep	30° – 45°	7	Clay soils – uniform gradational or duplex (including gley clays).	CL-CH or CH
			8	Precipitous slopes, cliffs, benched slopes, escarpment slopes etc.	> 45°	8	Clay soils or soils with very high liquid limit. (Clays, silty clays, volcanic ash soils etc.).	CH or MH
			9	Water ways.		9	Organic soils – peat, peaty clay etc.	OL-OH-Pt

EXAMPLE: GEOLOGY — TERRAIN MAPPING UNIT

ULTRAMAFICS — 75

GEOLOGICAL GROUP — PHYSIOGRAPHIC FEATURE — SOIL*

*NOTE: A dual symbol (1–5) indicates that both material types 1 and 5 are likely to occur within the unit as mapped.

Figure 5.3. Terrain map for construction materials investigation, Musa Dam, Papua New Guinea (courtesy of Coffey Partners International).

In horizontal strata mesa forms are common, and in dipping strata dip slopes and scarps indicate the direction and dip of the bedding (Fig. 5.1).

An important part of photo-interpretation for dam engineering is the recognition and plotting of lineaments. Lineaments are simply linear features or linear arrangements of features that are visible on the photographs.

Faults, particularly those which are steeply dipping, usually show up well as lineaments. A lineament indicative of a fault may be a linear arrangement of features which are topographically low, or otherwise indicative of the presence of deep soils or low strength materials. Such features include straight section of rivers or creeks, gullies, saddles, springs, swamps and unusually dense vegetation.

Figure 5.2 shows several lineaments passing through an area in Central Australia underlain by sandstones dipping to the east at about 5 degrees. The major lineaments occur along sheared zones with little or no displacement of the beds. The minor lineaments visible on the aerial photograph follow joints.

Linear arrangements of other features, e.g. vegetation boundaries, are usually found to correlate with geological features.

As discussed in Chapter 2, Section 2.6, it is very important that evidence of any currently active or past landsliding in the project area is recognised early. Fortunately the presence of active or ancient landslides is usually indicated by:

– unvegetated scarps, cracks and areas of exposed soil or rock;

– characteristic topographic forms, e.g. scarps, spoon-shaped troughs, hummocky or steep ground and areas of internal drainage;

– areas of anomalous vegetation, e.g. where trees are dead, or younger than elsewhere, or where the vegetation is more dense due to an area of deeper or wet soil;

– evidence of past or current restriction or damming of streams.

Rib and Liang (1978) provide guidance on the recognition of landslides on aerial photographs in a range of geological environments.

In densely vegetated areas evidence of active or very recent landsliding may not be readily visible on photographs. In some tropical areas shallow landsliding produces subdued topographic forms and revegetation is rapid. As shown in Figure 3.36 large landslides show up well even in tropical rainforest.

Another use of photo-interpretation is in terrain analysis where the features observed, including topography, geology, soils, drainage and vegetation are used to divide the area into land units with similar characteristics (Grant 1973, 1974). This type of study is more suited to regional assessment of catchment or reservoir areas, or to the location of construction materials, than to localised assessments, e.g. dam sites. Figure 5.3 is an example of a terrain map prepared to assist in the location of construction materials. Such maps form a logical basis for the planning of ground surveys followed by exploration of possible borrow areas. These maps should not be relied on for selection of sites without a field check of site conditions.

5.3 GEOTECHNICAL MAPPING

Geotechnical mapping is essentially geological mapping aimed at answering engineering questions. At any particular site it involves the location and plotting on suitable scales of all data which assists in understanding the geotechnical conditions at that site.

5.3.1 *Use of existing maps and reports*

Some useful data can often be obtained from existing maps and reports prepared for other purposes.

Maps showing the regional geology on scales ranging from 1:100 000 to 1:1 000 000 are usually available from government agencies. Some mining areas and areas of existing or proposed urban development may have been mapped at larger scales. The regional maps are often accompanied by explanatory notes which are useful.

The regional maps show the inferred distribution of the main rock types and the inferred geological structure. Areas where rock is overlain by unconsolidated sediments, surface soils or talus are not differentiated unless these deposits are known to be widespread and of significant depth. The data plotted on the maps is obtained from examination of surface outcrops, information from stratigraphic and exploration drilling, and air photo interpretation.

The main value of these maps is in providing an understanding of the stratigraphy, structure and geological history of the region, i.e. the broad geological environment in which the project is to be located. As set out in Chapter 3, this understanding can provide a useful insight into the range of geotechnical conditions which might be expected at the project sites. The maps are also useful during studies associated with access routes, material sources, and reservoir and catchment areas.

Local deposits of alluvial, colluvial or residual weathered materials are usually ignored or mentioned briefly. Because of this the maps generally do not answer detailed questions related to dam projects.

Geological reports prepared for different purposes, e.g. for mineral exploration, can provide some useful information.

Regional maps showing distribution of soil groups, classified for agricultural purposes, may also be available. These maps may be a useful supplement to the regional geology map, particularly in the consideration of construction material sources.

Land capability maps, based on the regional soil surveys with particular attention to potential landsliding and erosion, may be available for limited areas.

5.3.2 *Geotechnical mapping for the project*

5.3.2.1 *Regional mapping*

When published regional geological maps are available they are usually able to provide the regional geological understanding required for a dam project. For major dam projects, and for all projects where these maps are to be used in access, material or reservoir studies, these maps should be checked on the ground and on air photos. Where necessary they should be updated by the addition of data of engineering importance. Such data will include lineaments, landslides, scarps, swamps, springs and areas of problem soils.

If no satisfactory regional geological map is available, and the proposed dam is of major size and high hazard, then it would be prudent to prepare a regional map specifically for the project.

However produced the regional geological plan as used and included in the project reports should show the location of the proposed works and the outline of the proposed storage.

5.3.2.2 *Geotechnical mapping*

Geotechnical mapping at and near the sites of the proposed works is the key to the success of the site investigation. The mapping involves the identification and location of all surface features

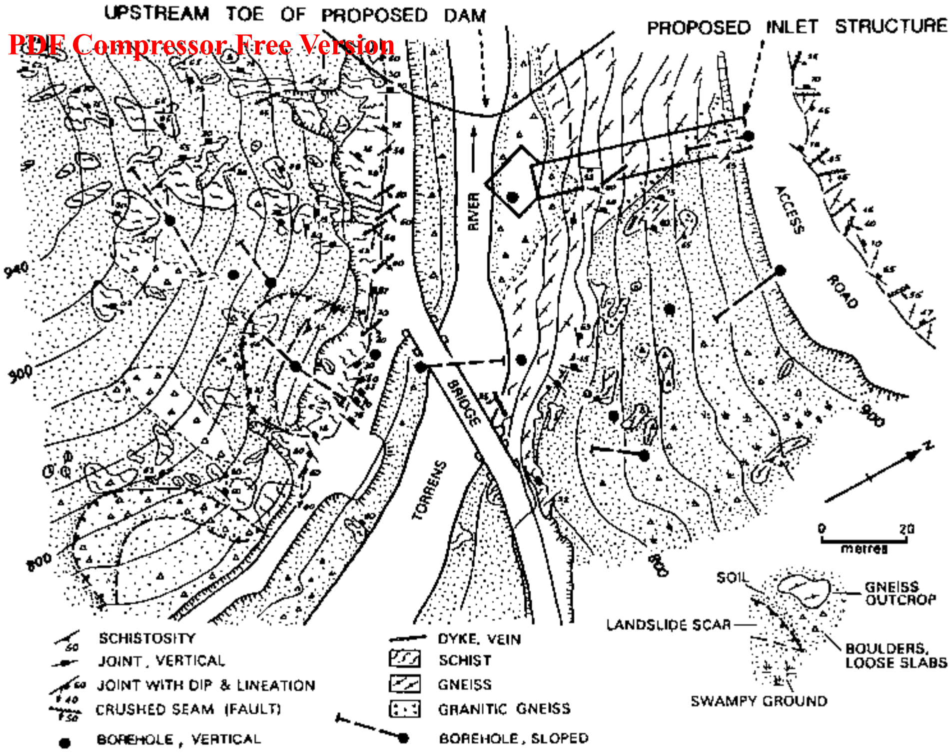


Figure 5.4. Part of detailed geotechnical map prepared during the planning of Kangaroo Creek Dam (courtesy of South Australian Dept. of Mines and Energy).

relevant to the establishment of geotechnical models at the sites. The plans and sections produced by the mapping form the initial geotechnical models which are the basis for the subsurface exploratory work aimed at checking the models, filling in gaps and answering specific questions raised by the indicated geological environments (see Chapters 3 and 4).

The geotechnical maps are usually produced at an intermediate scale (1:5000 or 1:2500) covering the general works sites, and at 1:1000 or 1:500 covering the immediate area of the sites. The maps show the following types of factual information as shown on Figures 5.4 and 5.5:

– ground surface contours;

– geomorphic features, e.g. slope changes, areas of hummocky ground;

– geological surface features, e.g. areas of rock outcrops, scree, boulders and soil;

– features of *in situ* rock, e.g. rock types and their boundaries, attitudes of bedding and foliation; the nature, location and orientation of important geological defects such as sheared or crushed zones;

– groundwater features, e.g. springs, seepage, areas of swamp and vegetation indicating moist or wet ground;

– the location of tracks, roads, test pits and trenches, together with the logging of soils and rocks exposed in these investigations;

– the position of drillholes and geophysical traverse lines;

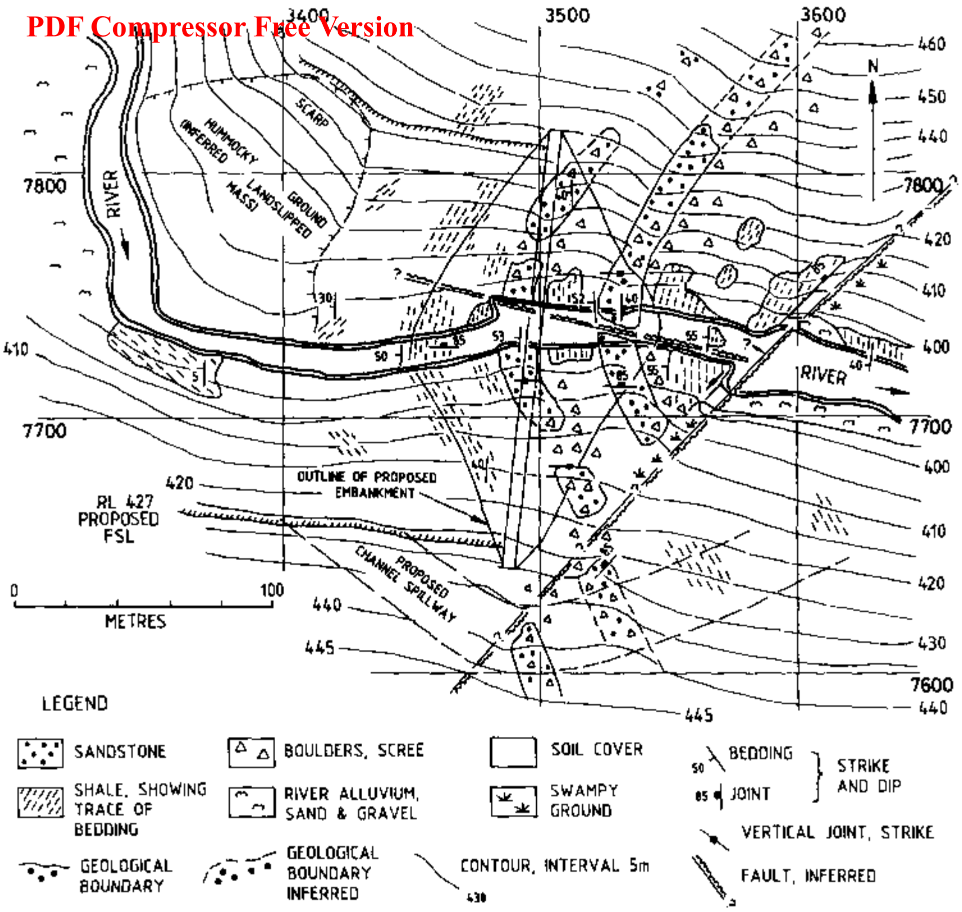


Figure 5.5. Plan showing some of the features presented on a large scale geotechnical plan of a site for an embankment dam.

– the proposed works in outline including the full supply level of the proposed storage.

Figure 5.5 is an example of the first stage of compilation of a geotechnical plan at the site for a concrete faced rockfill dam about 35 m high. At this stage the plan shows a factual record of surface geological and geomorphological features. Three relatively important features are inferred - the landslide upstream from the site and the two faults. Other important features might also be suspected, for example bedding-surface faults at the boundaries for the sandstone beds, because no actual shale-sandstone boundaries are exposed in outcrop, even at river level.

It should be noted that such a plan would normally be prepared on 1:500 or even 1:250 scale on an A2 or A1 size sheet. For the sake of legibility for publication at its present size the plan has been simplified greatly, i.e. it contains much less geological detail and wider contour spacing than would usually be present.

Position identification is important during the mapping. In sparsely vegetated areas the combination of enlarged air photos and contour plans at the same scale may enable positioning to an acceptable level of accuracy. Where required, more accurate control can be achieved by ground survey methods.

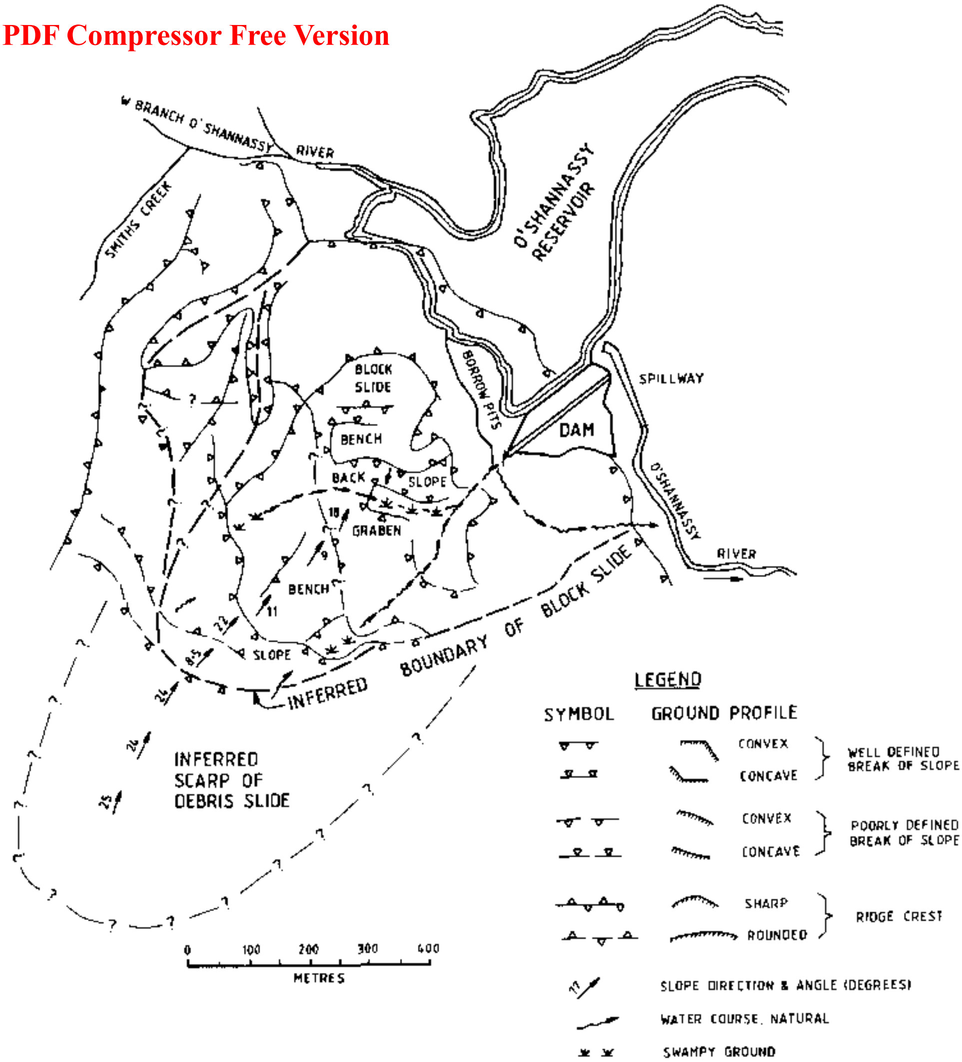


Figure 5.6. Large scale geomorphological plan of landslide area at O'Shannassey Dam, near Melbourne, Australia (courtesy Jeffery and Katauskas, and MMBW).

Geomorphological mapping may be useful in areas where the principal objective is the delineation of areas of past or present instability. It uses symbols to represent slope angles, slope shapes, erosion gullies and seepage areas as shown on Figures 5.6 and 5.7.

5.4 GEOPHYSICAL METHODS

Geophysical methods which have application to the investigation of sites for dams include:
– seismic refraction using both 'P' and 'S' waves,

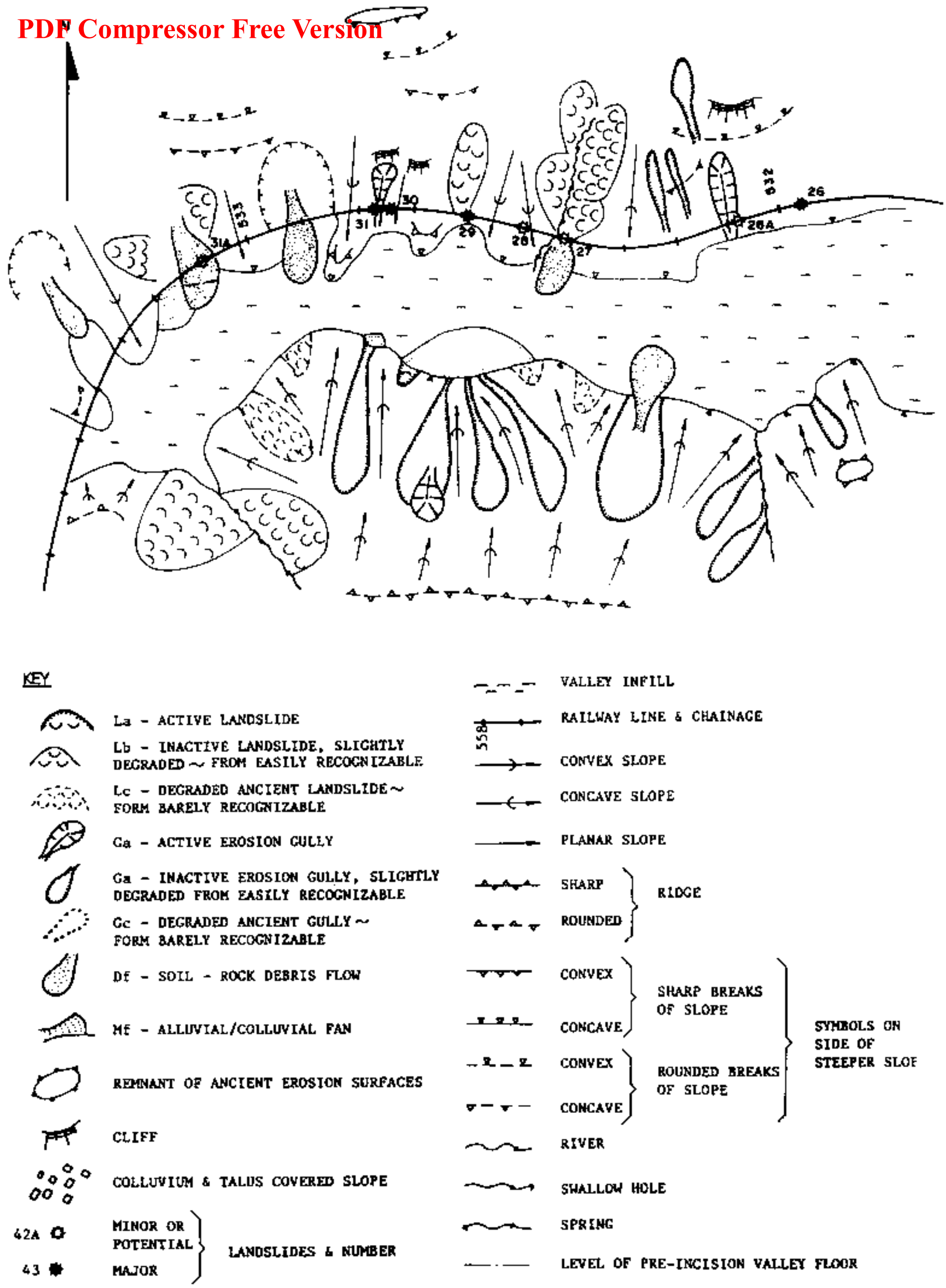


Figure 5.7. Small scale geomorphological map of TAZARA railway, Tanzania (MacGregor & Baynes, 1984).

– seismic reflection,

– electrical resistivity,
– ground probing radar,
– transient electromagnetic,
– downhole logging.

These methods are used as *in situ* testing tools to assist in:

a) Delineation of boundaries between features with contrasting physical properties. For example:

– boundaries between transported materials such as alluvium, colluvium, glacial debris, landslide debris and the underlying *in situ* rock;
– boundaries between residual soil, weathered rock and fresh rock;
– boundaries between sandy and clayey soils.

b) Location of features with anomalous physical properties. For example:

– igneous dykes or plugs;
– deeply weathered or altered zones;
– major fault zones;
– buried river or glacial channels;
– landslide masses;
– cavities.

c) Assessment of *in situ* physical properties of rock masses. For example:

– rippability;
– depth of general foundation excavation;
– depth of cutoff excavation;
– liquefaction potential of saturated sands and sandy gravel.

To be successful geophysical methods are usually in combination with other site investigation techniques. In many cases the purpose of the geophysical work is to check and assist in interpolation between data points established by other methods.

The different geophysical methods and their application to dam engineering are discussed briefly below with reference to papers containing detailed method descriptions. Further discussion is included in Whiteley (1983, 1988), Stapledon (1988) and Fell (1988).

5.4.1 *Seismic refraction 'P' wave*

This method utilises the fact that seismic waves travel at different velocities in different materials; in rock and soil masses the velocity increases with increase in substance strength and compactness. Whiteley (1988) provides details of the method and results.

Profiles of apparent seismic velocity are produced as shown on Figure 5.8, a section along the line of Sugarloaf Reservoir Inlet Tunnel.

Air photo interpretation had shown two lineaments crossing the line of the tunnel. Seismic refraction traverses, trenching and core drilling indicated that the lineaments are the surface expressions of a deeply weathered rock unit and a minor fault. This interpretation was confirmed during driving of the tunnel.

Figure 5.9 shows the results of a seismic traverse at the site for Kwae Noi Dam in Thailand.

Seismic refraction 'P' wave is the method most commonly used for delineation of boundaries between soil, and weathered rock, and within weathered rock profiles. It is the authors' experience that in a residual weathered profile the base of the lowest velocity layer is usually a reasonable approximation of the probable general foundation stripping level and the base of the

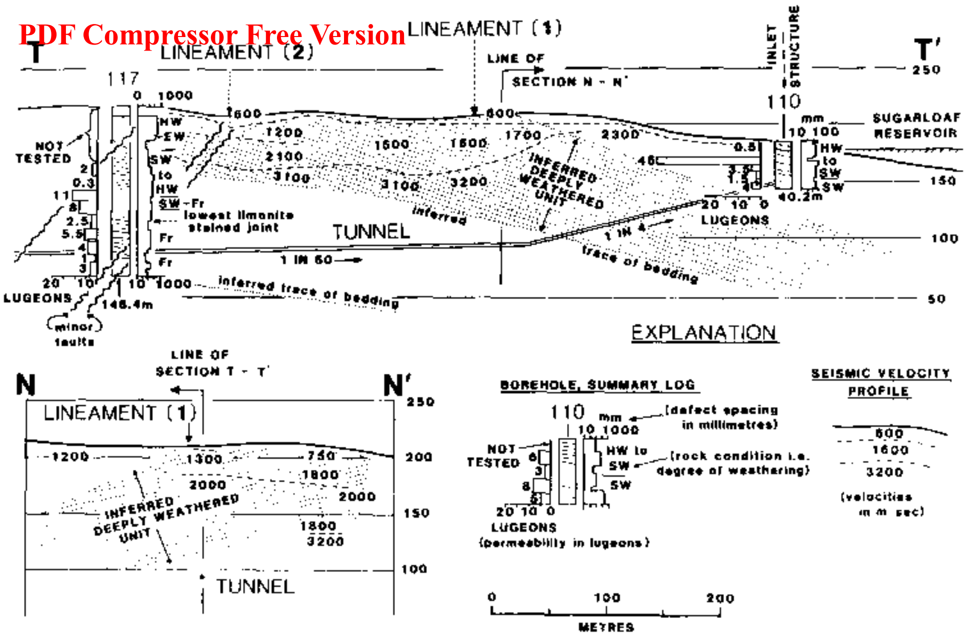


Figure 5.8. Section along Sugarloaf Reservoir Inlet Tunnel showing seismic refraction profiles and deeply weathered unit (Stapledon, 1988).

second layer a reasonable approximation of cutoff excavation level.

When combined with geological information the seismic 'P' wave velocities can be used to estimate the rippability of rock masses (Weaver 1975, Minty & Kearns 1983, Martin 1986, Braybrooke 1988, Fell 1988). Seismic velocities alone should not be used for predicting rippability.

Some limitations and requirements of the method are:

– The method cannot distinguish between sandy and clayey soils, and between soil and weathered rock where the boundary is gradational.

– The accuracy of the method is affected by poor velocity contrast between 'layers,' and where there are 'cliffs' in the rock profile.

– Accuracy can be improved by careful interpretation particularly avoiding averaging three layers into two.

– In layered and jointed rocks the seismic velocities usually show anisotropy which can be checked with cross traverses.

– The method cannot detect velocity inversions i.e. high velocity material above low velocity material, unless crosshole or downhole techniques are used.

– In loose and/or deep soils the method needs large energy input (usually explosives).

– Accurate ground survey is essential for efficient production of good results.

5.4.2 *Seismic refraction 'S' wave*

This method has been applied downhole to obtain shear-wave velocity profiles, from which dynamic and static Young's and shear moduli have been estimated. These values are used in

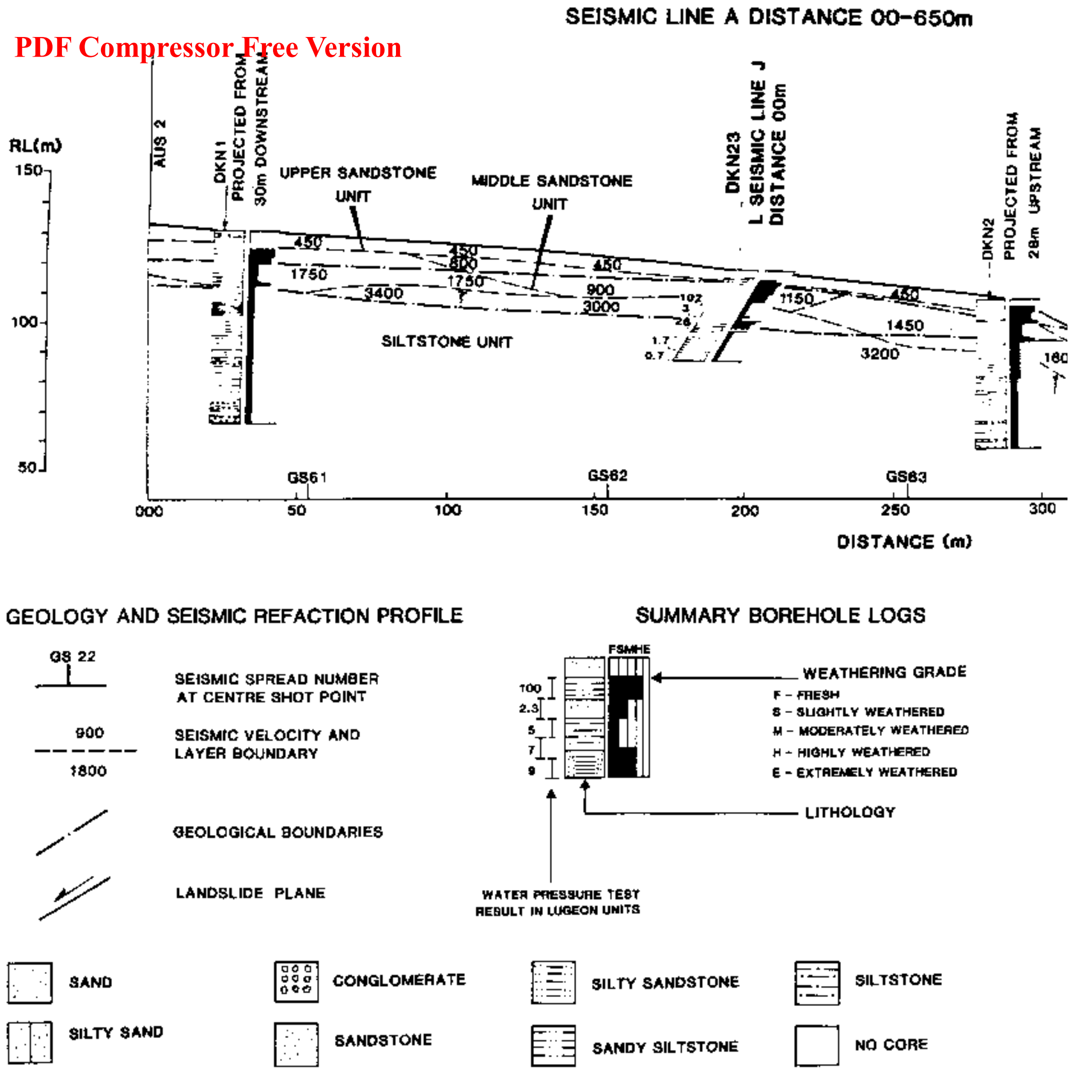


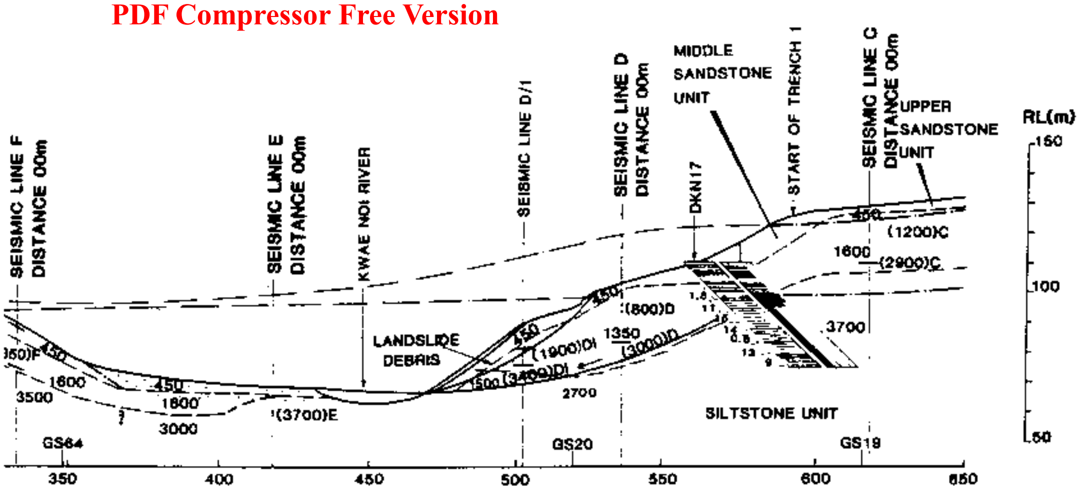
Figure 5.9. Seismic refraction survey results, Kwae Noi Dam, Thailand (Redecon, 1987).

dynamic analysis of the response of structures to earthquake loading. The shear-wave profile can be used to directly estimate the liquefaction potential of cohesionless soils (see Chapter 15).

Whiteley (1983, 1988) and Whiteley et al. (1990a) describe the techniques and applications in more detail.

5.4.3 *Seismic reflection*

This technique has had little use in dam engineering as it has been developed mainly for large scale exploration for oil, gas and coal. It is able to delineate layers within submerged soil



deposits (i.e. marine sediments) and will probably be applicable at the few dam sites these conditions occur.

Whiteley (1983) and King & Greenhalgh (1981) describe the technique.

5.4.4 *Electrical methods*

5.4.4.1 *Electrical resistivity*

The electrical resistivity method measures resistance of the ground to induced electric current. Interpretation assumes a horizontally layered model.

The method has been used to locate fault zones, zones of deep weathering and cavities (McCann et al. 1987, Smith & Randazzo 1987). It can also be used in the exploration of alluvial deposits where permeable gravel and sand beds can be distinguished from low permeability clays or rock. This capability has been applied in searches for construction materials beneath alluvial terraces and for foundation materials at dam sites where significant alluvial deposits occur.

At dam sites in valleys the results of resistivity surveys are affected by the irregular terrain and by changes in electrical properties between dry material on the abutments and the wet material beneath the valley floor. Depth-to-rock determination by resistivity in situations where the valley floor width is less than five times the expected thickness of alluvial material can be subject to significant side effects. These limitations and the generally poor depth resolution (about 10% of depth) indicate that the resistivity method has limited application in dam investigations.

Whiteley (1983) provides details of the method.

5.4.4.2 *Self potential method*

The self potential or SP method is based on measuring the natural potential difference which exist generally between any two points on the ground. The method has no application in most dam investigations. Erchul & Slifer (1987) have shown that anomalous potentials generated by flowing water may be useful indicators of sinkholes in karst terrain. This effect can also be used to locate areas of leakage from dams and reservoirs (Ogilvy et al. 1969, Bogoslovsky & Ogilvy 1972).

5.4.5 *Ground probing radar*

Ground probing radar (GPR) uses the ability of VHF electromagnetic (radio) signals to penetrate through soils and rocks. Usually the waves are radiated from an antenna passing along the ground. Reflections are received from features such as fractured zones and cavities in rock, and from distinct layers in soils. The method has also been used downhole.

The method has achieved some success in the location of sinkholes and cavities in karst areas, and this appears to be its main potential application in dam engineering. The results of surface surveys in karst areas are described by Wilson & Beck (1988) and Cooper & Ballard (1988). Cooper & Ballard (1988) also describe the location of cavities at the El Cajon dam site in Honduras, using a downhole technique.

Present limitations of the method include the requirement for a near flat surface and the restricted penetration through clayey soils.

Siggins (1990) and Morey (1974) describe details of the method.

5.4.6 *Transient electromagnetic*

The transient electromagnetic (SIROTEM) method produces resistivity – depth profiles (Whiteley 1983). The method is quicker and requires less space than resistivity, but may not be suited to the shallow depths normally associated with dam foundations.

5.4.7 *Downhole geophysical logging*

Values for electrical resistivity, self potential, gamma-ray emission and neutron absorption are routine downhole measurements during exploration for oil and coal. Most boreholes are uncored and the geophysical logging of each hole produces graphical plots from which the characteristic features of different lithological formations can be recognised (Whiteley 1983, Whiteley et al. 1990). Geophysical logging is relatively cheap but requires uncased and water-filled or mud-filled holes and has to be coordinated with the drilling programme.

In dam investigations boreholes are nearly all sampled or cored and downhole geophysical logging is not vital. The logging can assist in correlation between boreholes, location of potential leakage zones, and assessment of the properties of valley floor sediments. In the opinion of the authors specific geophysical logging is usually justifiable only for very large dams.

5.5 TEST PITS AND TRENCHES

5.5.1 *Test pits*

Test pits excavated by a rubber-tyred back-hoe or tracked excavator are effective in providing information on subsurface conditions, for the following reasons:

– They are relatively cheap and quick.

– The subsurface profile is clearly visible and can be logged and photographed.

– Material types, their boundaries and structure can be observed and recorded in three dimensions.

– The absence or presence of groundwater is indicated and the source of inflow can usually be observed.

– Undisturbed samples can be collected.

– *In situ* tests can be carried out.

– The resistance to excavation provides some indication of excavation conditions likely to be met during construction.

– It may be possible to leave pits open for inspection by design engineers and representatives of contractors. In this case fencing is usually required.

Figure 5.10 shows a log of a test pit which includes a section along one wall with descriptions of each material type, traces of their boundaries, and traces of defects. This section and the strip map showing the orientation of boundaries and defects enables the information to be transferred easily on to the relevant geotechnical plan and section.

Some limitations of test pits are:

– Pit depth is limited by the reach of the machine and its ability to dig the material. The maximum depth of a pit dug by backhoe is 4 m but the machine commonly refuses on very low strength rock. Larger excavators can reach 6 m and, depending on the rock defects, can excavate into low strength rock.

– Local laws on support requirements of excavations must be followed. Caving of pit walls occurs commonly, particularly in wet ground.

– Groundwater inflows may limit effective excavation, due to caving of the sides. It is sometimes possible to continue the pit below the water table but the main advantage of detailed observation of *in situ* material is lost.

– Test pits disturb the local environment, but if carefully backfilled and revegetated their long term impact is minor.

5.5.2 *Trenches*

A logical extension of the use of test pits is the excavation of trenches, using either a tracked excavator or tractor with bulldozer and ripper.

Following the slope movements which caused disruption of construction of Tooma Dam (as described in Chapter 2, Section 2.6), clear evidence of past slope movements were exposed as infilled extension features in cut faces of access tracks. It was concluded that this evidence would have been found, and probably understood, if the site investigation had included bulldozer trenches.

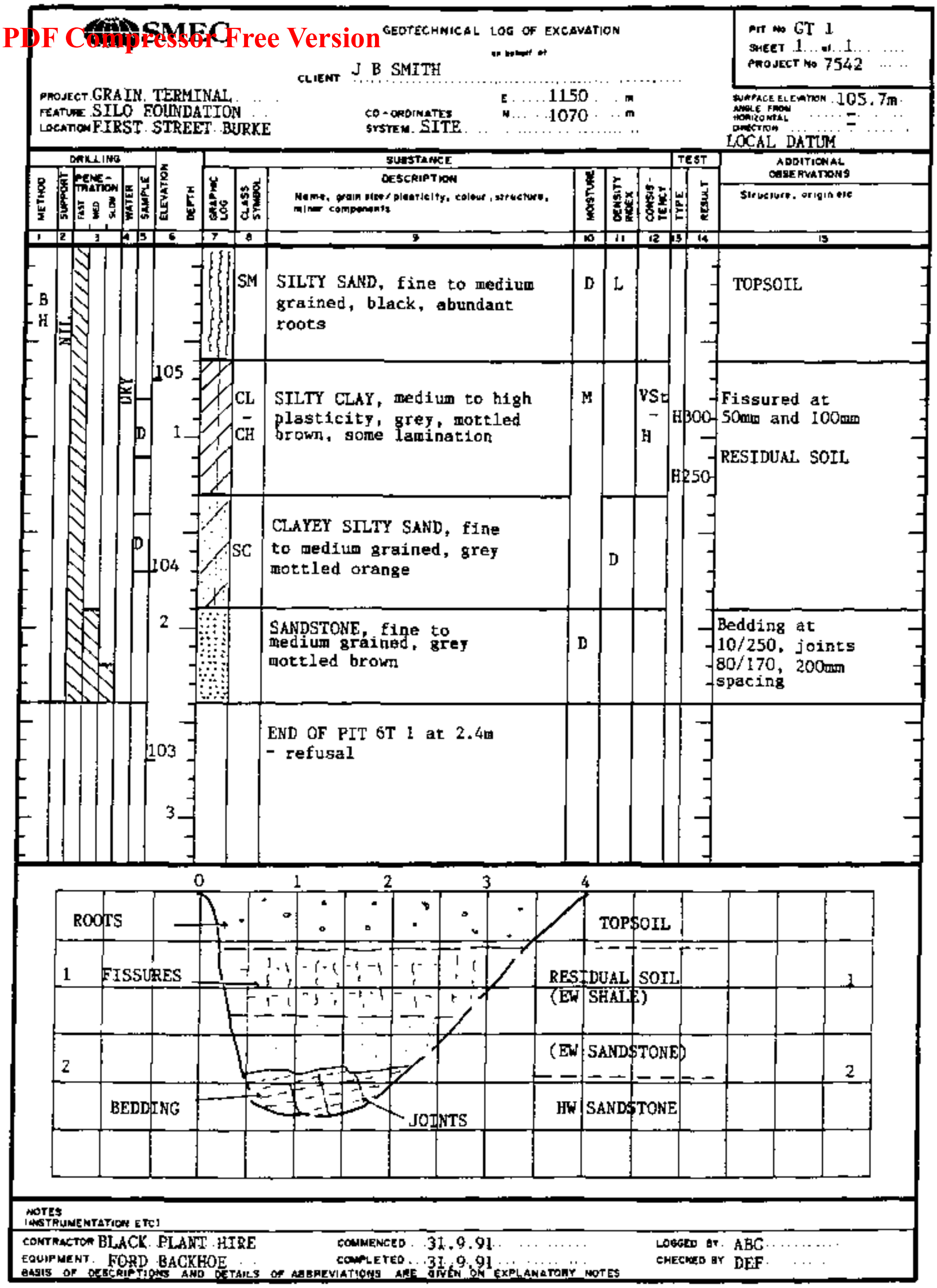
Following that example the sites for most dams built in Australia have been explored extensively by bulldozer trenches which have been designed to provide the answer to geotechnical questions and also access for drilling rigs.

These trenches have totalled more than 3 km in length at each of the following large embankment dams - Talbingo, Dartmouth, Sugarloaf and Thompson.

The effectiveness of well prepared trenches in providing an understanding of dipslope stability at the Sugarloaf site is described in Chapter 2, Section 2.6.3.2.

Bulldozer trenches provide virtually continuous exposures of the subsurface materials at sites where often there is little natural outcrop. Trench exposures are logged in a similar manner to test pits, i.e. in plan and elevation as shown on Figure 5.10.

A major benefit of trenching is the experience gained from the use of plant of similar size and type which may be used during construction. It is good practice to record machine and ripper types, material types and excavation rates.



SMEC — GEOTECHNICAL LOG OF EXCAVATION

on behalf of

CLIENT J B SMITH

PROJECT GRAIN TERMINAL
FEATURE SILO FOUNDATION
LOCATION FIRST STREET BURKE

CO-ORDINATES E 1150 m, N 1070 m
SYSTEM SITE

PIT No GT 1
SHEET 1 of 1
PROJECT No 7542

SURFACE ELEVATION 105.7m
ANGLE FROM HORIZONTAL –
DIRECTION –
LOCAL DATUM

Method (1)	Support (2)	Penetration (fast/med/slow) (3)	Water (4)	Sample (5)	Elevation (6)	Depth	Graphic log (7)	Class symbol (8)	Description: Name, grain size/plasticity, colour, structure, minor components (9)	Moisture (10)	Density index (11)	Consistency (12)	Test type (13)	Test result (14)	Additional observations: Structure, origin etc (15)
B H	NIL		DRY					SM	SILTY SAND, fine to medium grained, black, abundant roots	D	L				TOPSOIL
				D	105	1		CL – CH	SILTY CLAY, medium to high plasticity, grey, mottled brown, some lamination	M		VSt – H	H H	300 250	Fissured at 50mm and 100mm RESIDUAL SOIL
				D	104			SC	CLAYEY SILTY SAND, fine to medium grained, grey mottled orange		D				
						2			SANDSTONE, fine to medium grained, grey mottled brown	D					Bedding at 10/250, joints 80/170, 200mm spacing
					103	3			END OF PIT GT 1 at 2.4m – refusal						

NOTES (INSTRUMENTATION ETC)

CONTRACTOR BLACK PLANT HIRE — COMMENCED 31.9.91 — LOGGED BY ABC
EQUIPMENT FORD BACKHOE — COMPLETED 31.9.91 — CHECKED BY DEF

BASIS OF DESCRIPTIONS AND DETAILS OF ABBREVIATIONS ARE GIVEN ON EXPLANATORY NOTES

Figure 5.10. Log of test pit.

A disadvantage of trenching is that it causes much disturbance of the site environment. At the site for large structures in geologically complex areas with little natural exposure, systematic trenching is the most practical method of providing the answer to vital feasibility and design questions.

5.6 ADITS AND SHAFTS

In investigation for a major dam it may prove necessary to investigate a part or parts of the site, e.g. an area underlain by cavernous limestone or disturbed by a landslide, in more detail than can be achieved by the combination of surface excavations and drilling.

In such situations the excavation of adits or shafts into the area of concern provides the opportunity for:

- direct observation of the ground conditions
- measurement of orientation of defects and comparison with surface measurements
- *in situ* testing
- underground investigation drilling.

Exposures in adits and shafts should be logged and photographed in detail using the same general approach as for test pits and trenches. All exposed faces, including the floor if practicable, should be logged.

Adits and shafts are expensive and slow to excavate. They may require support, ventilation and drainage.

The requirement for, and location of, adits or shafts should be carefully considered in relation to all the available design and geotechnical information. It is necessary to justify the expense by the clarification of site conditions which they may provide. Wherever possible exploratory adits should be incorporated in the design of the dam, e.g. as drainage of grouting galleries.

Shafts using 1000 mm or larger diameter augers or clam shell excavators are often used to explore alluvial and glacial deposits.

5.7 DRILL HOLES

5.7.1 *Drilling techniques and their application*

Table 5.1 summarises the different drilling techniques available, their applicability, advantages and disadvantages.

The major advantage of drilling is that, subject to the choice of a suitable rig, there is little restriction on the depth to which the investigation can be taken. In the assessment of the subsurface profile, the properties of the lowest rock or soil unit which could affect the structure usually have to be determined. In many cases exploration by pit or trench will not reach this unit due to machine capacity, reach or groundwater inflows. In these circumstances drilling provides the most practical alternative.

Drilling also has little effect on the environment. Maximum hole size is usually less than 100 mm and can be easily covered or backfilled. Surface disruption is commonly restricted to the preparation of a drilling pad on sloping ground.

In many situations drilling and associated testing provides the only practicable method of

Table 5.1. Drilling techniques and their applicability.

Drilling techniques	Hole diameter	Support	Applicability	Advantages	Disadvantages and precautions	Approx. cost & productivity (1991)
Auger drilling Solid flight	Usually 100 mm	Self-support	Clayey soils, moist sand above the water table	Ease of setting up, rapid drilling, continuous disturbed sample recovered	Limited depth – 30 m normal maximum. Not applicable for sandy soils below water table because hole needs support	$15 per metre, 30 m/day
Auger drilling Hollow flight	Usually 150 mm	Self-support above water table. Augers used to support below water table	As above, and for sandy soils below water table by using augers for support, and sampling through centre of augers	As above, can be used below water table if flights filled with water or mud. Easy sampling	Hollow flights are normally not filled with water or mud so 'blowing' or disturbance at base of hole may occur particularly when withdrawing samples	$18 per metre, 20 m/day
Rotary drilling Non coring in soil	75-100 mm common	Self-support clayey soils. Casing and/or drilling mud (bentonite or chemical)	All soil types	Drill through all soil types. Allows full range of sampling and testing techniques	Poor identification of soil types in drill cuttings. Virtually impossible to identify thin beds of sand in clay, etc. Proper identification only possible from undisturbed or SPT samples. Unable to penetrate gravels efficiently. Must case and fill hole to surface with water or mud to prevent blowing in loose sand, and disturbance by shear failure in clay at depth. Note that stiff clays can fail by shear at 30 m depth if not supported by mud. Mud costly and takes time to set up and mix.	$30/m soil (75 mm), $5/m extra (100 mm), $50/m plus river gravel. Good soils investigations rotary drill rig with hydraulic top drive drill 3 m stroke. Variable speed head for augering/coring. Truck mounted with 2 or 3 winches to facilitate rapid removal of drill rods for testing. Hire rate is approx. $100 – 120 /h.
Rotary drilling Non coring in rock	As above	Usually no support in rock, may need mud in some rock	All rock types	Rapid drilling. However no sample recovered and rock identification difficult		
Rotary drilling Coring in soil	'N' O.D. 76 mm, core 52 mm. 'H' OD. 100 mm, core 76 mm. EX, AX, BX should not be used in soil.	Casing and drilling mud. May use air or foam to improve recovery	Clayey and silty soils. With care good recovery can be achieved in all soils with any cohesion.	Continuous 'undisturbed' sample allows identification of stratification in alluvial soils.	No recovery in clean sand and silt. Samples usually disturbed by stress relief and swell due to water used for drilling. 'H' size core gives better recovery but is more difficult to handle due to added weight.	NMLC coring $60-80 per metre. HLC $70-100 per metre. 10-15 m/day coring. Best carried out on hourly basis to ensure best practicable core recovery. Rig hire $100-200 per hour. Add $5 /m for bit wear.

Rotary drilling Coring in rock		Usually no flush with water	All rock types	Continuous core of rock		
Reverse circulation	Usually BQ Duo-tube	Outer casing of reverse equipment	Sands, sand with fine gravel	Rapid drilling. Complete recovery of soil drilled (disturbed). Useful for investigation of sand sources	Less commonly available. More expensive equipment. Mixes strata. No testing or undisturbed sampling possible	Rig hire $100-120/hr plus compressor hire. Up to 30 metres per hour
Cable tool percussion	Up to 750 mm	Casing which may or may not be recovered	Sands, gravel	Can penetrate gravels. Can drill large diameter hole. Simple equipment. Often available in remote areas used for water bores	Slow drilling. Gravel broken up to allow recovery in bailer can give misleading particle size distribution. Strata mixed in breaking up and recovery process. Fines often lost in drilling process – 'Sandy gravel' logged as 'gravel'. Very heavy casing and tools for transport and handling.	$60-90 per hour. 2 m per hour down to 0.10 m per hour
Hammer drill (impact drill)		Self supporting in rock	Rock which will support	Very rapid penetration rate (200 m in 24 hours eg.)	Samples limited to rock chips. May lose equipment in loose fractured rock	$60-90 per hour plus compressor hire
Bucket auger, casing and auger, casing and clam shell	900 mm to 1200 mm	Casing which may or may not be recovered	Sands and gravels	Drill through gravelly soils without breaking up into smaller sizes. Suitable for investigation of sand/gravel filter sources	Costly. Sub-sampling and testing also expensive because of large sampling size	$250 per metre for holes to 10 m. 10 m/day
Rotary drilling with rig on all terrain vehicle	As above	As above	Drilling in swamps, water up to 0.5 m deep	Allows access where truck mounted rig would not enter because of soft ground	Smaller drill rig mounted. Applicable to 30 m depth	Bombardier $200 per day additional to ordinary drilling cost per day
Rotary drilling with rig on small jack-up barge	As above	As above	Drilling in water up to 6 m deep. Larger rigs available but essentially for offshore use	Float rig into position and set on bottom, unaffected by tide or waves	Costly to establish. Smaller drill rig mounted. Applicable	$500 per day additional to ordinary drilling cost. Add cost of work boat if applicable

Other costs: U50 (Tube only) $16; U75 (Tube only) $30 + $25 to take (above 30 m); SPT $25 each to 30 m; Core boxes $6/metre, establishment costs (and accommodation if applicable).
Notes: NMLC – 76 mm borehole, 52 mm core; 'H' – 100 mm borehole, 76 mm core.

determining the permeability of the rock in the dam foundation.

Drilling has the disadvantage that information obtained is almost always indirect – either from the observation of resistance to rig penetration, by the measurement of *in situ* properties with equipment lowered down the hole or by the logging of samples recovered by the drilling. Direct observation is restricted to the use of mirrors, down-the-hole television and the logging of large diameter auger holes.

Drilling is also relatively slow - rates depend on the machine and the material type and typically are 4 to 5 auger holes to 10 m in soil per 8-hour shift or 15 m of core drilling and water pressure testing per 8-hour shift. Rigs are often truck mounted – which makes them mobile but restricts access in sloping ground. Skid mounted rigs require considerably more time for establishment at each site.

Because of the expense and time involved in drilling it is not highly cost-effective, but it is the only practical method of obtaining samples and direct data on permeability at depth.

It is not good practice to specify drilling programmes in detail before geotechnical mapping and logging of test pits and/or trenches is well advanced or completed. Boreholes drilled without the understanding provided by there activities can be regarded as 'wildcats'.

Each borehole should be carefully planned, using the progress geotechnical model, to answer specific questions, or to fill in gaps in the model. The borehole objectives should be explained in the specification provided to management and to the driller.

During the drilling programme the results of each completed borehole are used to further upgrade the geotechnical model, which is then used to plan subsequent holes.

5.7.2 *Auger drilling*

The most common type of drilling in cohesive soils and rock of soil strength uses a spiral flight auger to penetrate and remove the material below the surface. The simplest form is the hand auger which is usually restricted by the physical effort involved to about 3 m.

Most augers are machine driven - and range from portable to truck-mounted hydraulic drill rigs (Figure 5.11). This drill rig is also suited for rotary drilling non coring (wash boring) and for diamond drilling.

The common auger rig equipped with either solid or hollow flight augers, can reach up to 30 m in soil strength materials. A steel blade 'V' bit will penetrate most fine-grained soils and extremely low strength rocks but usually refuses on coarse gravel or low strength rock (Table 2.4). A tungsten-carbide 'TC' bit will grind slowly through low and medium strength rock.

Large diameter pier boring machines can drill holes 1 m in diameter up to 10 m deep – provided no major wall support problem is encountered.

Auger drilling allows the logging of disturbed material collected from the flights during drilling. By removal of the augers it is practical to regularly recover tube samples and carry out *in situ* testing of the material properties.

A major difficulty in auger drilling in cohesionless soils is the stability of the sides of the drill hole particularly below water. Figure 5.12 shows a log prepared from an augered drill hole in sandy soils below the water table, compared with the log of a nearby static cone penetrometer probe. Mixing and collapse of the hole have led to gross inaccuracies in the auger hole log. Auger drilling in non-cohesive soils is therefore restricted to above the water table.

Hollow flight augers have the advantage of providing support for the hole in these conditions and allow sampling through the augers. However the action of removing the plug at the end of the auger will often loosen and disturb the soil below the auger.

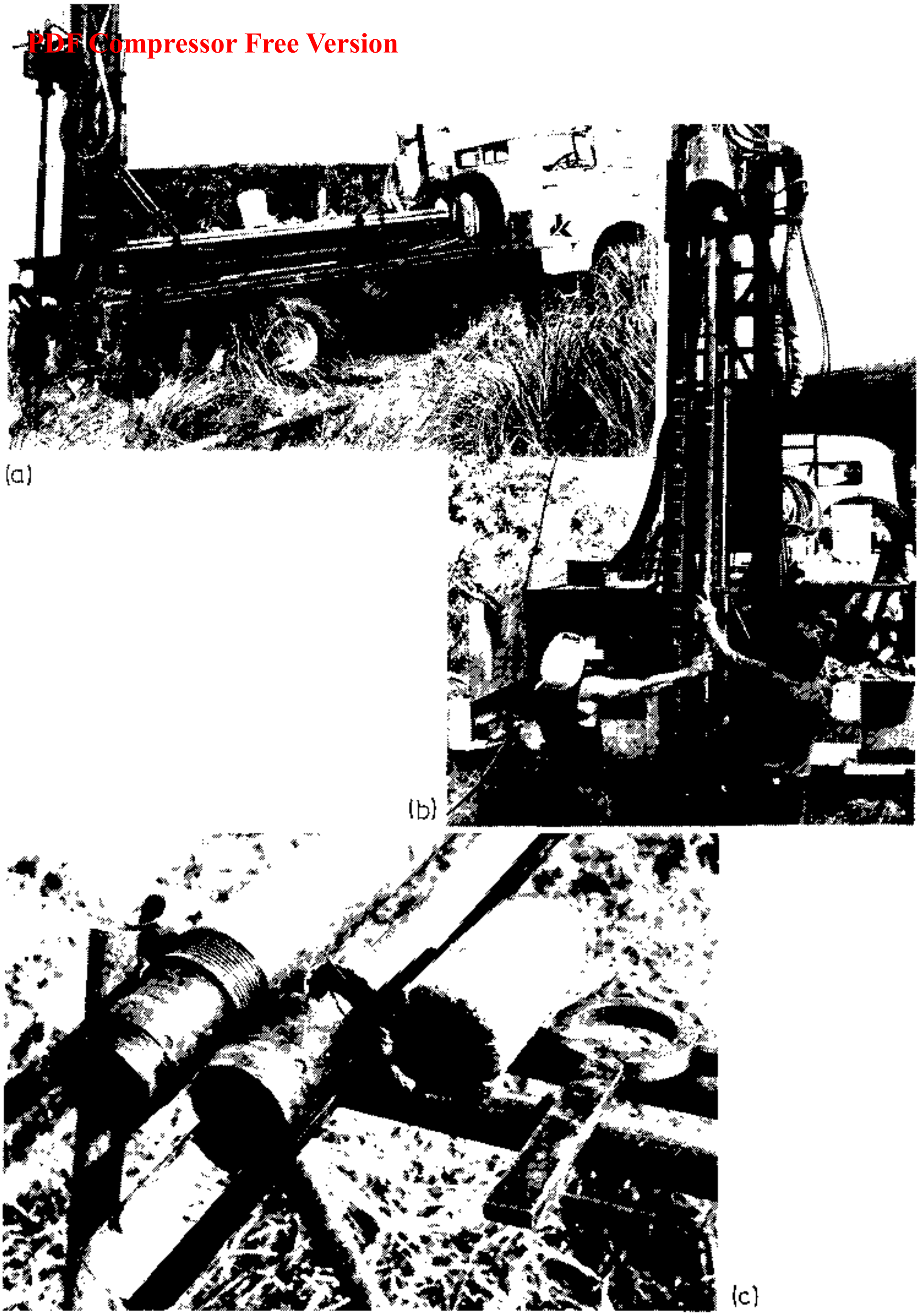


Figure 5.11. Rotary drilling rig a) Side view. Mast can be tilted to drill angled holes. b) Rear view showing top drive hydraulic motor. c) Dismantled core barrel showing inner tube, core catcher and drill bit (courtesy Jeffery & Katauskas).

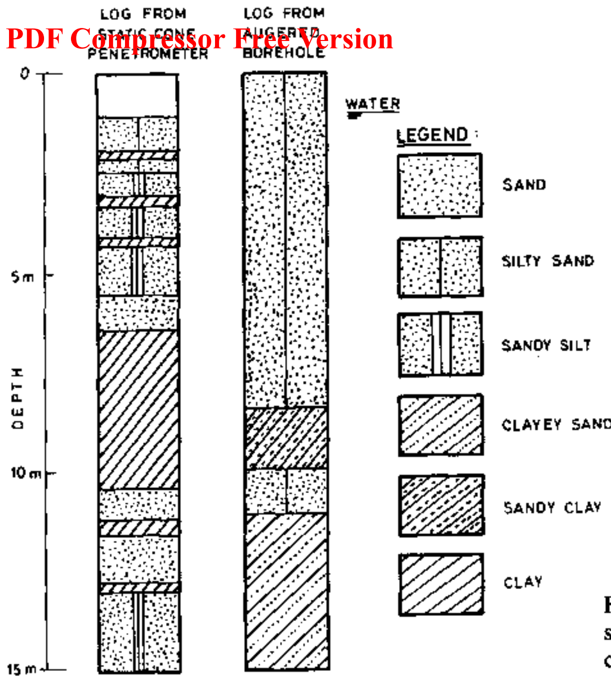


Figure 5.12. Log of sandy and clayey soils below the water table a) from static cone penetrometer, b) from auger drilling.

Auger drilling is suited to the investigation of areas with thick soil deposits which extend beyond the practical limit of pits or trenches. It does not provide the same amount of data on soil structure but can supplement other information. In many cases it is used as a rapid method of establishing the depth and general properties of the material overlying rock which will be investigated in more detail by some other method.

5.7.3 *Percussion drilling*

Penetration of rock or gravelly material beyond the capacity of an auger can be achieved by breaking up the material into small fragments. There are two main methods:

a) Cable tool drilling (Fig. 5.13) involves the successive dropping of a heavy chisel type bit to the bottom of the hole. The fragments are recovered using a bailer. If the hole is dry, water is added to aid the bailing. As the hole is advanced steel casing is driven to preserve the hole. The method is slow, but is effective in penetrating gravels and should be restricted to that purpose. It should be appreciated that the penetration process involves a reduction in particle size and mixing of material from several layers. The grading of samples recovered from cable tool drilling may be different from that of the natural material.

b) A compressed air driven hammer drill similar to an air-track used in quarry operations can provide rapid penetration of medium to high strength rock. Results are restricted to measurement of penetration rates and logging of the sand or gravel sized rock fragments driven to the surface by the circulating air. This method provides no data on rock structure. The efficiency of the drilling is affected by excessive groundwater and also the drilling is slowed by the blocking of the bit vents by clay.

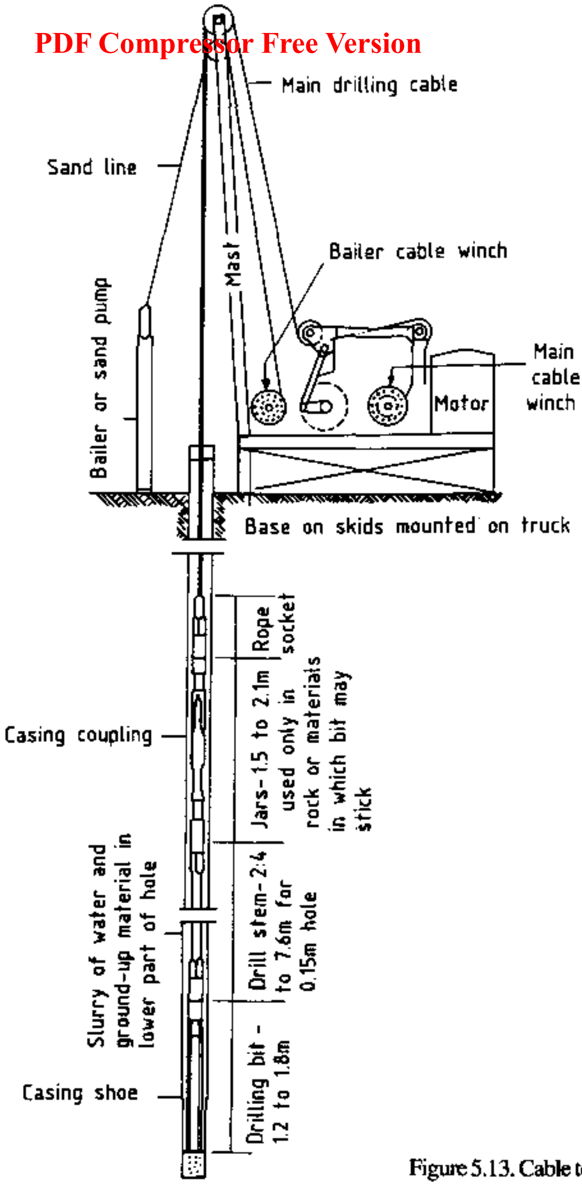


Figure 5.13. Cable tool rig.

It is considered that air percussion drilling has little or no application in dam investigations, but can be useful as a supplement to core drilling in the investigation of overburden depths and rock quality at proposed quarry or spillway channel sites.

5.7.4 *Rotary drilling*

Drilling using a rotary machine (Figs 5.14 and 5.15) in soil and rock can involve either core or non-core methods.

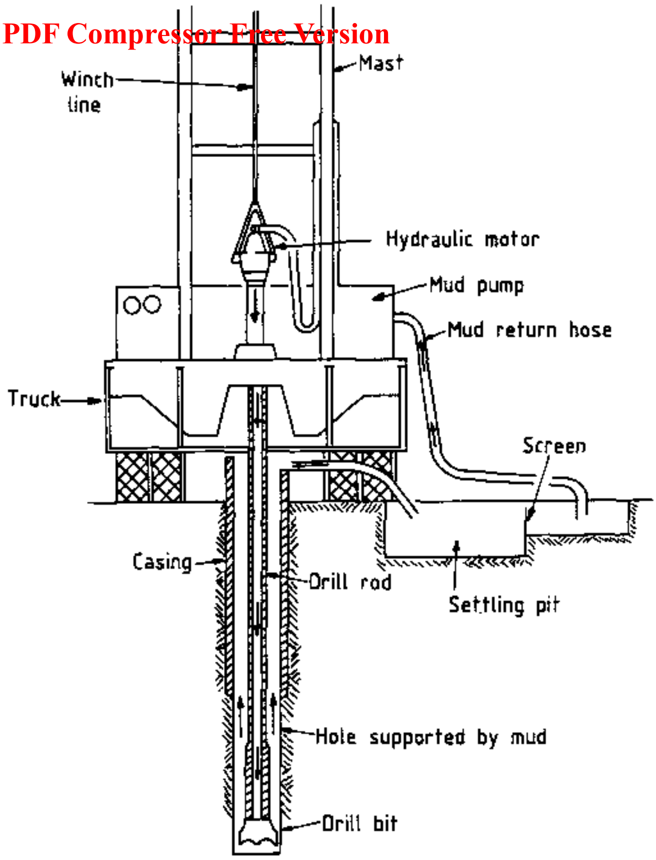


Figure 5.14. Rotary drilling rig – non coring.

a) Non-core drilling, sometimes called wash boring, involves the use of a solid roller, button or drag bit at the end of drill rods. The bits break up or grind the full face of the bottom of the hole and the fragments are removed by the circulating fluid - commonly water. Samples from non-core rotary drilling through soil and rock are a slurry of water and fine-grained material. This leads to poor identification of the materials.

It is usual to collect thin-walled tube samples of cohesive soils for more accurate material description and laboratory testing and to carry out *in situ* testing in the borehole.

Non-coring bits can penetrate rock and care must be taken to change to coring methods as soon as resistance to penetration is encountered.

When drilling in sandy soils above the water table and in most soils below the water table, drilling mud is used to support the hole sides and assist in the recovery of drill cuttings. Either bentonite or more commonly chemical mud is used to form a cake on the sides of the hole (Fig. 5.16). By maintaining a head of mud above the water table the excess pressure supports the sides of the hole. Bentonite mud must remain dispersed to be efficient. In saline groundwater the bentonite may flocculate and reduce the ability of the mud to support the sides of the hole.

Some chemical muds can be designed to maintain their viscous nature for a limited time and revert to the viscosity of water after a few hours. These muds have the advantage of not affecting the permeability of the material surrounding the hole and are suited to holes where permeability testing is required.

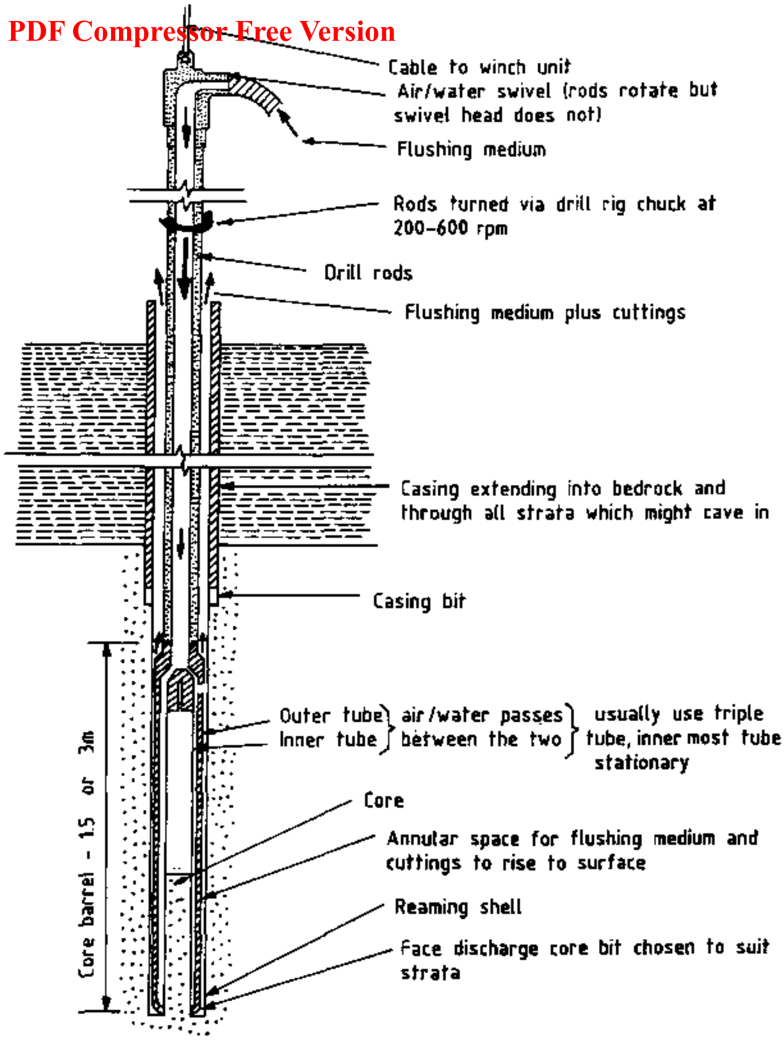


Figure 5.15. Rotary drilling rig – coring.

Chemical drilling muds can be used to improve recovery when coring in weak, erodible rock or stiff to hard soils. Despite the 'reverting' nature of the mud, the authors experience is that permeabilities obtained by water pressure testing in holes drilled with mud are lower than those obtained in holes drilled with water.

Core recovery can also be improved by using chemical foam instead of water or mud. This is described in Brand & Phillipson (1984).

b) Core Drilling - An annulus of rock is removed using a hollow bit with a leading edge impregnated with fragments of diamond or tungsten carbide, leaving a cylinder, or core, of rock which can be removed by the core barrel. In conventional core drilling (Fig. 5.15) the barrel is connected to the end of drilling rods which convey the circulating fluid to the bit. Each time the core barrel is filled the rods have to be removed.

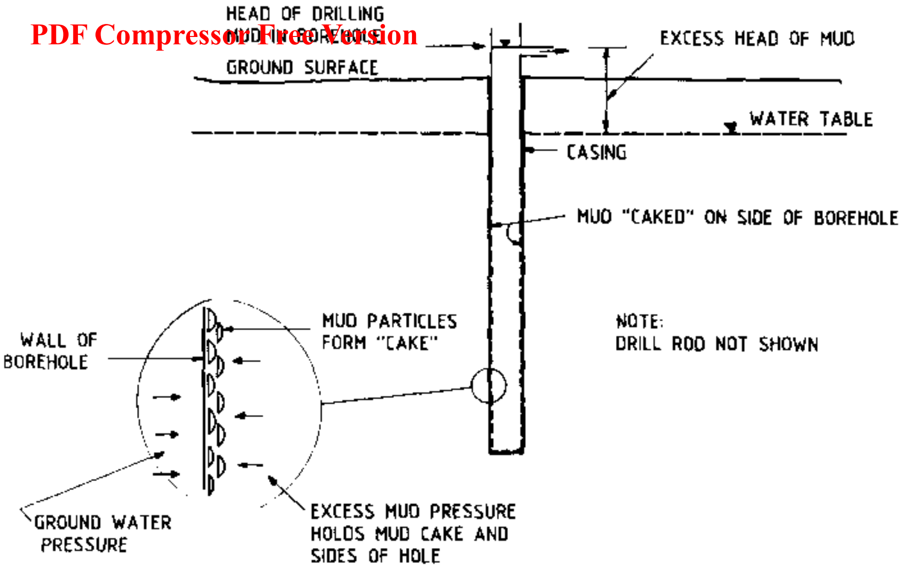


Figure 5.16. Principle involved in use of drilling mud to support sides of a borehole.

In wireline drilling the use of thin walled rods and a special core barrel enables the core to be raised inside the rods and significantly reduces the time involved in drilling deep holes. The core recovered by wireline is slightly smaller in diameter than for conventional drilling (NMLC core 52 mm, NQ core 47 mm).

The objective of site investigation drilling is to obtain the maximum amount of information on subsurface conditions. Every effort is required to recover as much core as possible. In low strength, fractured rock this may involve many short runs with low thrust and drilling water pressures. Triple tube, stationary inner tube, core barrels (Fig. 5.17) reduce core disturbance, improve core recovery and should be used in dam investigations.

In the analysis of foundation conditions, particularly in relation to slope movement and potential leakage, the nature of the low strength material is most important. A drilling programme which recovers all the strong rock and loses all the weak material is ineffective and its results may be misleading. As a matter of course drillers should be required to comment on the probable reason for every section where core has not been recovered.

Drilling contracts should be worded to ensure effective core recovery rather than rapid drilling progress. The loss of 1 m of core in the upper part of the hole may be much more important than the recovery of 10 m of fresh core at the base of the hole.

Placement of the core in boxes and logging of the core should emphasise sections where core was not recovered as these may represent crucial zones in the foundation.

The choice of the core size to be used has to be considered in relation to the ability to provide full core recovery. Larger diameter core is more expensive to drill, but is likely to produce a higher core recovery in low strength and fractured rock.

The ratio between natural fragment size in the rock to be sampled and core diameter is important. It is almost impossible to core conglomerate with 50 mm pebbles with N size equipment (75 mm hole, 52 mm NMLC core, or 47 mm NQ core). As a general rule coarse

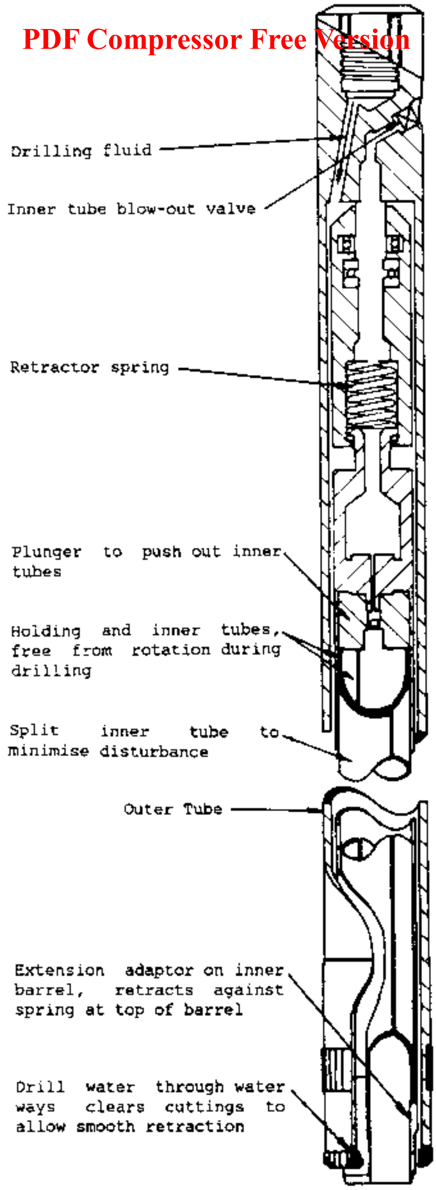


Figure 5.17. Triple tube core barrel (NMLC) (Triefus).

grained fractured rock requires large diameter core. N size drilling is commonly used for high-strength rocks, but H size (100 mm hole, 63 mm HMLC core, or 63 mm HQ core) is significantly more effective in lower strength materials and only costs about 10% more than N size.

On many dam sites defects which control strength and permeability in the dam foundation are close to vertical. Vertical drill holes are unsuitable for sampling these features and in the assessment of their effect on the foundation permeability.

The ability to drill angled holes at selected azimuths enables the choice of hole direction to provide maximum information including the orientation of rock defects. Angled holes are commonly slower to drill and slightly more expensive (contract rates are usually 10 to 15% higher than for vertical drilling). Drill holes at angles of less than 45° to the horizontal require anchoring of the rig to provide thrust.

5.8 SAMPLING

5.8.1 *Soil Samples*

Soil samples recovered from the subsurface site investigation are described as either disturbed or undisturbed.

Disturbed samples are collected from pits, trenches and from auger flights as representative of different material types or units. They are identified by location and depth and stored in sealed containers (usually plastic bags) for laboratory testing. For cohesive soils about 3 kg of sample is required for classification testing and 30 kg for compaction tests to evaluate probable performance as engineered fill in an embankment.

Undisturbed samples consist of material which is extracted from the site and transported to the laboratory with a minimum of disturbance. The ideal sample is a cube of approximately 0.3 m sides, hand cut from a test pit, carefully packed and sealed on site and transported to the laboratory without delay.

It is more usual, due to economic factors, and the limitations of depth of test pits, to use thin-walled steel tubes ('Shelby' tubes)to obtain samples of cohesive soils from boreholes. The tubes are pushed into the soil using an adaptor connected to the drill rods. Care should be taken to ensure that the drill hole is cleaned out before sampling and that the drilling water or mud level is maintained during sampling to avoid 'blowing' of material into the hole. Choice of sample diameter depends on the hole size but, in general, the larger the sample diameter the less the disturbance. Common thin walled tube sizes are 50, 63 and 75mm. The wall thickness and cutting edge shape are defined by codes to limit sample disturbance. Samples should be identified and sealed against moisture loss using either a sample tube sealing device or several layers of molten wax. Undisturbed samples should preferably be tested within two weeks of sampling as they rust into the tube, or despite all efforts, dry out. On extrusion in the laboratory a proportion of 'undisturbed' samples often prove to have been partly disturbed by the sampling process and it is prudent to take enough samples to allow for this.

In any case, even apparently undisturbed samples are affected by stress changes during sampling, and this needs to be recognised when analysing the results.

5.8.2 *Rock Samples*

Samples of rock exposed in the sides/floor of pits/trenches can be taken for testing of substance strength and mineralogy. Samples should be individually numbered and located. Storage in plastic bags prevents loss of field moisture.

The most common form of rock sample is core. A drill hole with full core recovery should present a complete linear profile through the rock mass below the ground surface. It is important that the core is systematically stored, properly logged, photographed and sampled as soon as practicable after drilling.

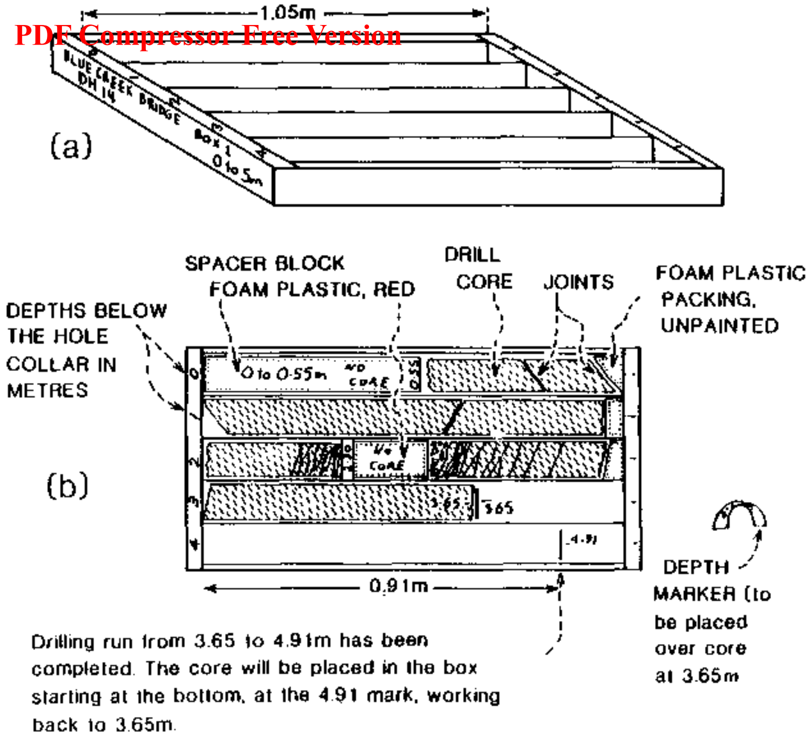


Figure 5.18. Systematic boxing of drill core. a) Empty core box labelled ready for use b) Partly filled box during drilling.

In most cases the core needs to be kept for the following purposes:

– To enable the site investigator to make an accurate, clear and concise log of those characteristics of the core which are significant to the project; and to use this log in the compilation of the geotechnical model.

– To enable the project designers, and tenderers, to examine the core and assess those characteristics of most significance to the project.

– To provide samples for testing.

The characteristics of most significance to the project will usually be:

– the material strength
– the lengths of the drilling runs
– the lengths of individual pieces of core
– the lengths and positions of core losses
– the length, position and engineering character of important defects such as sheared or crushed zones
– the engineering character of joint surfaces
– the depths at which significant changes in rock substance type or strength occur.

The systematic method of boxing core is illustrated by Figure 5.18 and Table 5.2. This method 'reconstructs' the core in the core box, to scale, packed with all joint or broken faces fitting, without any spacers apart from sections of core loss. The box is constructed in compartments of

Table 5.2. Good practice in boxing drill core.

Item	Description
Tray length	1.05 m internal to hold 1.0 m of core and allow for 5% increase in length due to bulking or oblique breakage
Boxing scale	Core is boxed to scale in the ground; each row in the tray represents 1 m of hole length. Any core loss is filled by a block of timber or equivalent, painted red, the length of the core loss and positioned where the core is considered to have been lost. If the position is uncertain the block is placed at the top of the run
Boxing layout	Core is stored in the trays like the lines of writing on a book.
Box marking	The left hand external end of the tray is identified with the following information: Project, hole no., box no., depths. The same information should be recorded inside the box. The drilled depth in metres from the surface is marked on the left hand end of each row. Detachable core tray lids should not be marked
Core marking	The bottom end of the core in each run is marked by paint or a 'permanent' felt pen with the depth of the hole and a line at that depth is marked on the bottom of the tray. A depth marker (see Fig. 5.18) is placed over the core at the bottom of each run with the depth recorded on the marker. Runs should not be separated by spacers. Where the core tends to break along oblique layering, or where core is very broken, the full 1.05 m may be required to store the 1 m of core drilled. Where the core has few joints and can be broken normal to the axis the extra 50 mm allowed for bulking may not be needed and can be filled with while plastic foam to keep the core tightly packed and avoid disturbance
Core samples	Core samples are removed only by authorised persons and are replaced immediately by yellow blocks of equivalent length marked with the name of the person who removed the core, the date of removal and reason for removal

a convenient length - usually either 1 or 1.5 m – which become the 'scale' for use by the driller and others who examine the core.

Alteration in moisture content can produce significant changes in substance strength. Representative samples should be sealed, as wrapping in the thin plastic film used for food is only effective in the short term.

Core can deteriorate rapidly on exposure and should be stored inside in covered boxes. This is a particular problem for some shales and siltstones which can slake to soil within several days of sampling.

5.9 *IN SITU* TESTING

In many cases it is preferable to measure the properties of soil and rock in dam foundations using *in situ* tests, rather than taking samples and testing in the laboratory. In some cases (e.g. estimation of the relative density of sands) *in situ* testing is the only method available.

The topic is wide and rapidly developing and this section is restricted to directing readers to the available literature.

5.9.1 *In situ testing in soils*

The most commonly used *in situ* tests in soils are:

– Standard penetration test.

– Static cone penetrometer and piezocone.

– Vane shear.
– Pressuremeter.
– Dilatometer.
– Plate bearing test.

Jamiolkowski et al. (1985) and Wroth (1984) give overviews of the topic, and more recent conferences, e.g. Penetration Testing 1988, ISOPT-1 (de Ruiter 1988), and Pressuremeter Testing (ICE 1989) contain state of the art papers on the individual methods.

The main methods and their applicability to dam engineering are:

a) Standard penetration test (SPT). Refer to Decourt et al. (1988), Skempton (1986) and Nixon (1982) for details.

The test is useful to obtain estimates of the relative density (density index), effective friction angle and deformation modulus (E) of cohesionless soils, and to assess the liquefaction potential of saturated sands and silty sands. The test is widely available but non-standardised and inherently approximate.

b) Static cone penetrometer (CPT) and piezocone (CPTU). Refer to De Beer et al. (1988), Campanella & Robertson (1988), Jamiolkowski et al. (1988) and Kulhawy & Mayne (1989) for details.

The tests are useful to obtain estimates of the relative density, effective friction angle, deformation modulus (E) of cohesionless soils, and the undrained shear strength of soft cohesive soils.

The CPT and CPTU are particularly useful in alluvial foundations where sandy soils are interlayered with clayey soils as the instrument is able to detect the layering better than most drilling and sampling techniques. This test can also be used to assess the liquefaction potential of saturated cohesionless soils.

The CPT test is widely available but non-standard. The CPTU test is less widely available and requires some corrections to allow for the different designs of equipment. Both are inherently approximate but are generally regarded as more precise than the SPT test.

c) Vane shear. Refer to Bjerrum (1973), Aas et al. (1986), Walker (1983), Azzouz et al. (1983) and Wroth (1984) for details.

This test can only measure the undrained shear strength of very soft to firm clays (maximum undrained shear strength of about 70 kPa), and has little use in dam engineering. The need for correction of the field vane strength to give design strengths is well documented in Bjerrum (1973), Azzouz et al. (1983) and Aas et al. (1986).

d) Pressuremeter. Refer to Ervin (1983), Jamiolkowski et al. (1985), Wroth (1984), Campanella et al. (1990), Powell (1990) and Clarke & Smith (1990) for details.

Both the self-boring (SBPM) and Menard type pressuremeters have application in dam engineering, particularly in the assessment of the compressibility of soil and weathered rock foundations. These pressuremeters give the most accurate estimate of compressibility provided that the test method and interpretation is correct.

The instruments are able to estimate the effective friction angle of cohesionless soils, and the undrained shear strength of cohesive soils. SBPM test results can also be used to estimate the *in situ* horizontal stress in the ground, an important factor in modelling deformations in dam foundations using finite element methods, and to establish *in situ* stress conditions for dynamic analysis of liquefaction. The tests are relatively expensive (in comparison to CPT and SPT) but affordable on most large dam site investigations.

e) Dilatometer. Refer to Kulhawy & Mayne (1988), Lacasse & Lunne (1988), Powell & Uglo (1988) for details.

Table 5.3. Perceived applicability of *in situ* test methods (Wroth 1984).

	Soil type	Profile	Density D_r	Angle of friction ϕ	Undrained shear strength c_u	Pore pressure u	Stress history OCR	Modulus: Young's and shear (E and G)	Compressibility m_v and C_c	Consolidation c_h and c_v	Permeability k	Stress-strain curve	Liquefaction resistance
Dynamic cone	C	A	B	C	C	–	C	–	–	–	–	–	C
Static cone													
Mechanical	B	A	B	C	B	–	C	B	C	–	–	–	B
Electrical friction	B	A	B	C	B	–	C	B	C	–	–	–	B
Electrical piezo	A	A	B	B	B	A	A	B	B	A	B	B	A
Electrical piezo/friction	A	A	A	B	B	A	A	B	B	A	B	B	A
Acoustic probe	C	B	B	C	C	–	C	C	–	–	–	–	C
Dilatometer	B	A	B	C	B	–	B	B	C	–	–	C	B
Vane shear	B	C	–	–	A	–	B	–	–	–	–	–	–
Standard penetration test	B	B	B	C	C	–	–	–	C	–	–	–	A
Seismic cone penetration test downhole	C	C	C	–	–	–	–	A	–	–	–	B	B
K_o blade	–	–	–	–	–	–	B	–	–	–	–	–	–
Resistivity probe	B	B	A	B	C	–	C	C	C	–	–	C	A
Borehole permeability	C	–	–	–	–	A	–	–	–	B	A	–	–
Hydraulic fracture	–	–	–	–	–	B	B	–	–	C	C	–	–
Screw plate	C	C	B	C	B	–	B	A	B	C	C	B	B
Seismic downhole	C	C	C	–	–	–	–	A	–	–	–	B	B
Impact cone	C	B	C	C	C	–	C	C	C	–	–	–	C
Borehole shear	C	C	–	B	B	–	C	C	–	–	–	C	–
Ménard pressuremeter	B	B	C	B	B	–	C	B	B	–	–	C	C
Self-boring pressuremeter	B	B	A	A	A	A	A	A	A	A	B	A	A
Self-boring devices													
K_o meter	–	–	–	–	–	–	A	–	–	–	–	–	–
Lateral penetrometer	C	C	B	B	B	–	B	B	C	–	–	–	–
Shear vane	B	C	–	–	A	–	B	–	–	–	–	–	–
Seismic cross-hole	C	C	B	–	–	–	–	A	–	–	–	B	B
Nuclear tests	–	–	A	B	–	–	–	C	–	–	–	–	C
Plate load tests	C	C	B	B	C	–	B	A	B	C	C	B	B

*A, high applicability; B, moderate applicability; C, limited applicability.

The (Marchetti) dilatometer can be used in a similar way to the CPT. Its proponents claim advantages over the CPT but these are not apparent to the authors at this time.

f) Plate bearing tests. Refer to Pells (1983), Powell (1986) and Powell & Quarterman (1988) for details.

Plate bearing tests on the surface, in pits or downhole can be useful in assessing the deformation modulus of soil and weathered rock in a dam foundation. However as the tests are usually restricted to near the ground surface they are of limited value in a situation with a thick compressible layer.

Table 5.3 reproduced from Wroth (1984) summarises the applicability of the different *in situ* tests. The authors broadly agree with the table.

5.9.2 *In situ testing of rock*

The most common used *in situ* tests in rock are:

- water pressure permeability test.
- pressuremeter.
- plate bearing.
- borehole orientation.
- borehole impression.

The water pressure test is described in Section 5.12. The references given in Section 5.9.1 for pressuremeter and plate bearing tests cover testing in rock as well as in soil.

a) Borehole orientation. Refer to Hoek & Bray (1981) for details.

The logging of samples of soil and rock recovered from drill holes can identify the nature and spacing of defects and orientation in relation to the axis of the hole (more commonly expressed with reference to the plane normal to the core axis), but the absolute orientation is required for analysis of the pattern of defects which affect the project.

There is no reliable method by which defects in core from any vertical drill hole can be accurately oriented. In specific cases fractures can be related to distinctive features (e.g. bedding) which have been oriented on surface or underground exposures. In most cases orientation by this method is at best an educated guess. A recent development has been the use of remanent magnetism to orient drill core from Coal Measure Rocks (Schmidt 1991). This technique involves the establishment of the regional orientation of remanent magnetism. Specific samples can then be oriented in relation to this direction.

It is possible to orient core from an angled drill hole using several different methods.

The simplest method applies when a hole is angled through rock in which the orientation of distinct bedding, foliation or cleavage is known, and is believed to remain constant throughout the depth of the hole. A length of core is simply held in the same orientation as the borehole, and then rotated until the bedding, foliation or cleavage lies in its known orientation. The orientation of joints or other defects can then be measured by compass and clinometer.

Other methods require down-the-hole instruments such as the Craelius core orientation device (Hoek & Bray 1981).

Provided that the core is not disturbed or excessively broken, it is usually possible to orient several of the defects in each core run.

Analysis of the results of these measurements should consider the inherent problems involved. Accuracy of better than 20° in azimuth is difficult.

b) Borehole impression device. This instrument records defects on the sides of the drill hole (Fig. 5.19). It involves a tube with an expandable rubber packer, within foam-covered split

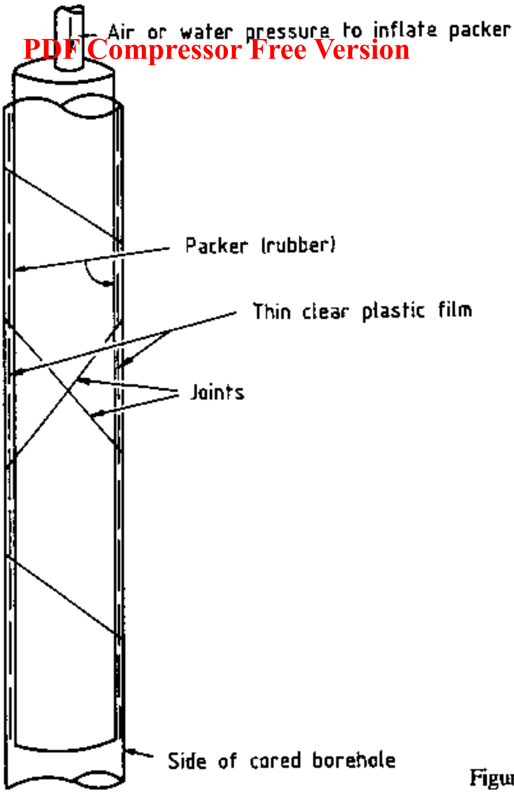


Figure 5.19. Borehole impression device.

metal leaves and a sleeve of thermoplastic film. When the packer is inflated – by water or gas – an impression is retained on the film of the defects which are present on the side of the hole.

If the impression device is oriented (using marked drill rods), an impression survey should provide orientation data on defects from both vertical and angled drill holes.

In practice it has been found that successful use of the impression device has been limited to materials of medium to high strength with widely spaced open joints. Closed defects may not form a sufficient indentation in the wall of the drill hole to show on the sleeve. Tearing of the thermoplastic film is common in fractured rock.

5.10 GROUNDWATER

In all aspects of the investigation for embankment dams the groundwater situation should be continually monitored. This involves the location and monitoring of seepage points during surface mapping, recording of groundwater inflows into pits and trenches, and the monitoring of groundwater levels during the drilling investigation. Piezometers (with sealed tips and not just open slotted PVC 'wells') should be installed in drill-holes (see Chapter 18) and regularly measured throughout the project. The response of groundwater levels to rainfall can provide a useful indication of mass permeability.

Claims related to unforeseen groundwater conditions form a significant proportion of contractual disputes. Many of these claims originate from a failure to record adequate groundwater information during site investigations, and during the period between the completion of investigations and the start of construction.

Measurement of seepage and flow rates, recording of rainfall, evaporation, and water levels installed in exploration drill holes gives a broad indication of the proportion of rainfall which infiltrates, the response of the groundwater to storms, and areas where high groundwater flow rates may be expected. This programme provides a relatively low cost general picture of the regional hydrogeology which can be refined by an investigation of specific permeability values at individual sites.

5.11 *IN SITU* PERMEABILITY TESTS ON SOIL

The permeability of soil in a dam foundation may be of importance if the cutoff is founded in the soil, as in dams on alluvial, colluvial and glacial deposits and on some deeply weathered residual or lateritised soils.

In general the structure of the soil controls water flow, for example:

– sandy layers in alluvial soils,
– root holes and fissures in residual soils,
– worm burrows in alluvial and residual soils,
– leached zones and relict joints in lateritised soil and extremely weathered rocks.

Hence it is seldom possible to obtain a realistic estimate of soil mass permeability from laboratory testing. Some of the difficulties associated with laboratory testing are discussed in Chapter 6.

Because the soil structure tends to be blocked or smeared by the drilling action, pump-in type permeability tests in boreholes can also give quite misleading results, with measured permeabilities one or two orders of magnitude lower than the actual soil mass permeability.

Where possible pump out tests should be conducted with a pump well and observation wells. Figure 5.20 shows a typical arrangement.

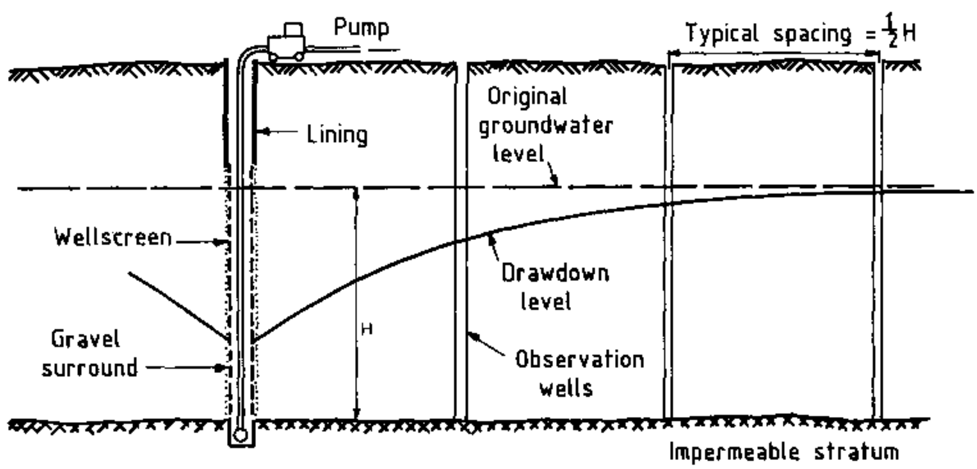


Figure 5.20. Typical arrangement for a well pumping test with observation wells.

In a pump out test the inflow of water removes the soil blocking or smearing the hole side, and realistic estimates of permeability can be obtained.

Design of the pump well, observation wells and the test program analysis need to recognise the effect of different vertical and horizontal permeabilities in the soil. The results are often difficult to assess and it is recommended that a groundwater hydrologist be employed in the planning, execution and analysis of such tests.

In the event that the groundwater level is below the depth of interest, it is necessary to carry out pump-in tests. Even when considerable care is taken to clean the hole it should be understood that the results of such tests give lower bound estimates, and that the actual permeability may be up to one or two orders of magnitude greater than indicated by the test result.

5.12 *IN SITU* PERMEABILITY TESTS IN ROCK

5.12.1 *Lugeon value and equivalent rock mass permeability*

The permeability of the rock mass can be determined by either constant head tests or falling head tests. A section of a drill hole is isolated using a sealing packer and water is added to maintain a constant head, or the rate of fall in water level is measured after a slug of water is added to the hole. Both methods suffer from the potential effects of smear and clogging of defects, but by careful flushing of the hole before testing, reasonable values can be obtained. In falling head tests the additional pressure which can be added to the test section is limited by the level of the test section and the practicability of extending the pipe imposing the head above ground level.

The most common and effective method of measuring rock mass permeability is the water pressure test (also known as the Lugeon or 'packer' test). The test consists of isolating a section of drill hole and pumping water under pressure into that section until the flow rate for any given pressure is constant (i.e. it is a constant head test). The use of successive rising and falling test pressures establishes the relationship between the volume of water accepted into the section and the pressure to provide an estimation of permeability, and indicate water flow mechanisms.

As rock substance is generally almost impermeable the permeability determined in this test represents an indication of the number, continuity and opening of the rock defects which intersect the wall of the borehole in the test section.

Results are expressed in Lugeon (uL) units. A Lugeon is defined as the water loss of 1 litre/minute per metre length of test section at an effective pressure of 1 MPa.

Indicative rock permeabilities are:

Lugeon	Range	Condition
< 1	Low	Joints tight
1-5	Low/Mod.	Small joint openings
5-50	Mod/High	Some open joints
> 50	High	Many open joints

There is no unique relationship between Lugeon value and equivalent rock mass permeability (ke). Moye (1967) recommended use of the equation

$$\mathrm{ke} = \frac{\mathrm{QC}}{\mathrm{LH}}$$

where ke = the equivalent coefficient of permeability (m/sec);

Q = the flow rate (m^3/sec);

L = the length of the test section (m);

H = Net head above the static water table at the centre of the test section (m);

$$C = \frac{1 + \ln(L/2r)}{2\pi}$$

r = the radius of the hole (m).

This is based on the assumption of radial laminar flow in a homogeneous isotropic rock mass, a condition seldom, if ever, achieved.

Hoek & Bray (1981) suggest the use of the equation

$$ke = \frac{Q\ln(2mL/D)}{2\pi\, LH}$$

where $m = (ke/kp)^{1/2}$

kp = equivalent permeability parallel to the hole

ke = equivalent permeability normal to the hole

D = diameter of hole.

They suggest that for most applications ke/kp is about 10^6. This implies no fractures parallel to the hole and in most rocks would not be a reasonable approximation. If it is assumed that ke/kp = 10, and water pressure testing in 5 m lengths of NMLC hole (75 mm diameter), then 1 Lugeon is equivalent to ke = 1.6×10^{-7} m/sec. For ke/kp = 1, i.e. homogeneous, isotropic conditions, m = 1 and 1 Lugeon is equivalent to ke = 1.3×10^{-7} m/sec. The Moye (1967) formula gives similar results to this homogeneous isotropic case.

5.12.2 *Test methods*

There are two common methods of water pressure testing in a drill hole (Fig. 5.21). The 'down-stage' (or 'single-packer') method is recommended and involves isolating and testing successively the bottom sections of the drill hole. This method enables progressive assessment of permeability and allows later stabilisation of the wall of the hole by casing or grouting if caving occurs. A disadvantage of down-stage packer testing is that it disrupts drilling progress, but this is far outweighed by its advantages over the alternative methods discussed below.

The alternative method is to complete the drilling of the hole and water test in sections by sealing the hole above and below the test area (the 'double-packer' method). This method has the advantage of convenience in that all water testing is carried out at one time but results can be affected by:

– Damage to the sides of the hole by drill rods and casing .

– Possible leakage from the test section past the lower packer, which cannot be detected.

– Sections of the hole which have been stabilised by cement or by casing to enable deeper drilling cannot be tested. Commonly these sections will be highly permeable.

The purpose of the test is to estimate the potential of water to pass through rock defects. The use of drilling mud to stabilise the hole can block these defects and make the results of water pressure testing meaningless. For best results the drilling fluid used in holes where permeability testing is required should be water. If necessary a small amount of soluble oil appears to improve drilling efficiency without affecting permeability.

There are several proprietary chemical drilling muds such as 'Revert' which are reputed to

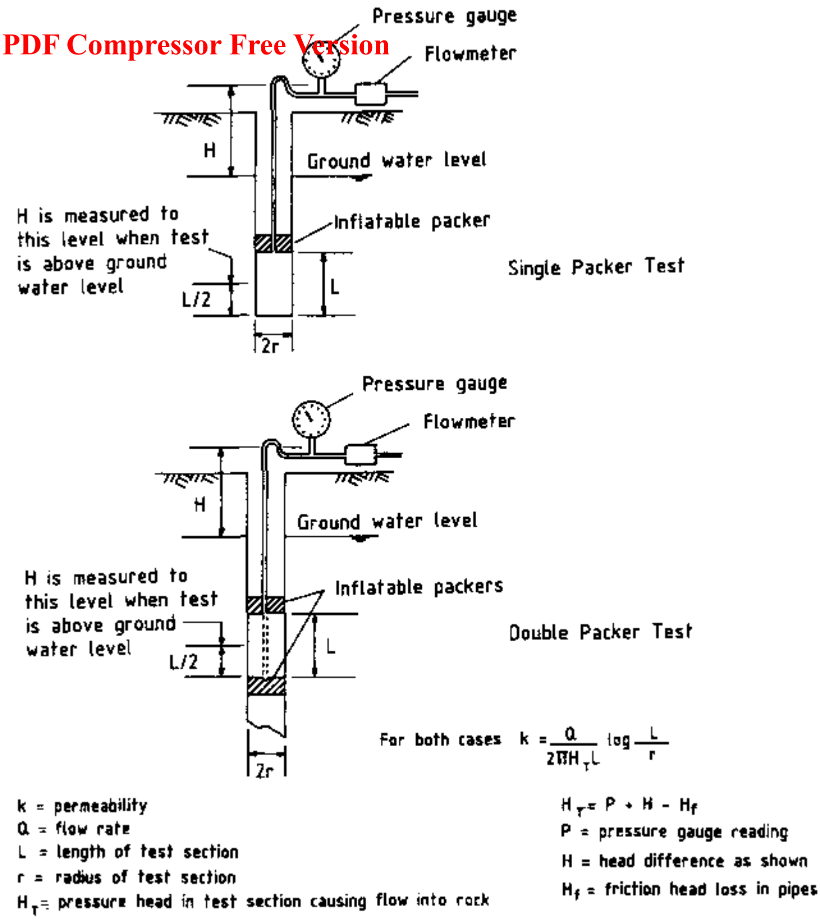


Figure 5.21. Packer permeability testing; single packer and double packer methods.

break down and dissolve when treated with 'Fastbreak'. Testing experience indicates that this procedure is only partly effective and that holes drilled with Revert give lower indicated permeabilities than holes in the same situation drilled with water.

5.12.3 *Selection of test section*

Every effort should be made to test the total length of hole in rock. It is preferable to overlap sections and thus have two tests over a short length than to miss some length of hole.

The upper limit which can be tested is the highest level at which a packer can be satisfactorily sealed, often in distinctly weathered rock. Location of the packer within casing above the rock does not seal the hole as water may leak past the casing.

The length of test section depends on the nature of the rock defects and the type of structure under investigation. Examination of the drill core usually indicates the presence of typical fracture spacing which represents probable background permeability and anomalous structural features which may be associated with zones of higher permeability.

Test sections should be selected to provide an indication of the relative width and permeability of these zones. In many cases water flow may be concentrated through a few fractures. The water loss is averaged over the length of the test section and, if the section is too long, the presence of a high permeability zone may not be recognised.

Usual test section lengths range from 3 to 6 m but may be increased in essentially unfractured rock. When a particularly high water loss is recorded it is good practice to repeat the test over a shorter section of the hole to further define the zone of high permeability.

5.12.4 *Test equipment*

The equipment required for water pressure testing includes:

– a packer to seal the test section,
– a water line from the packer to the supply pump,
– a pump to supply water under pressure,
– a bypass to control the pressure,
– a water storage,
– a pressure gauge,
– a water meter.

A layout used for water pressure testing is shown on Figure 5.22.

5.12.4.1 *Packers*

There are several types of packer used:

– A hydraulic packer (Fig. 5.23) consists of a double tube with rubber sleeve. When the packer is in the 'down' position water can be pumped into the sleeve to inflate the packer and seal the hole. In the 'up' position water flows into the test section.

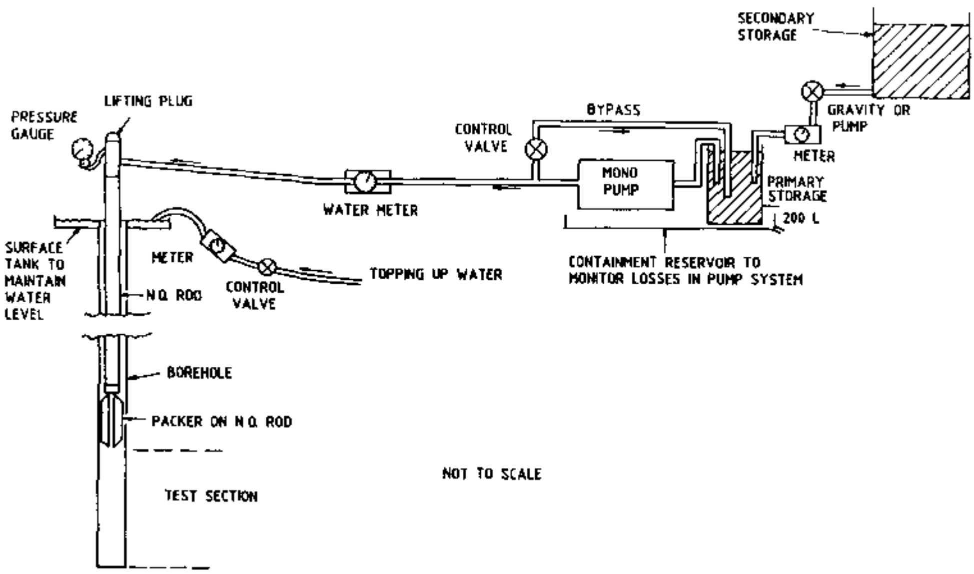


Figure 5.22. Layout of packer test equipment.

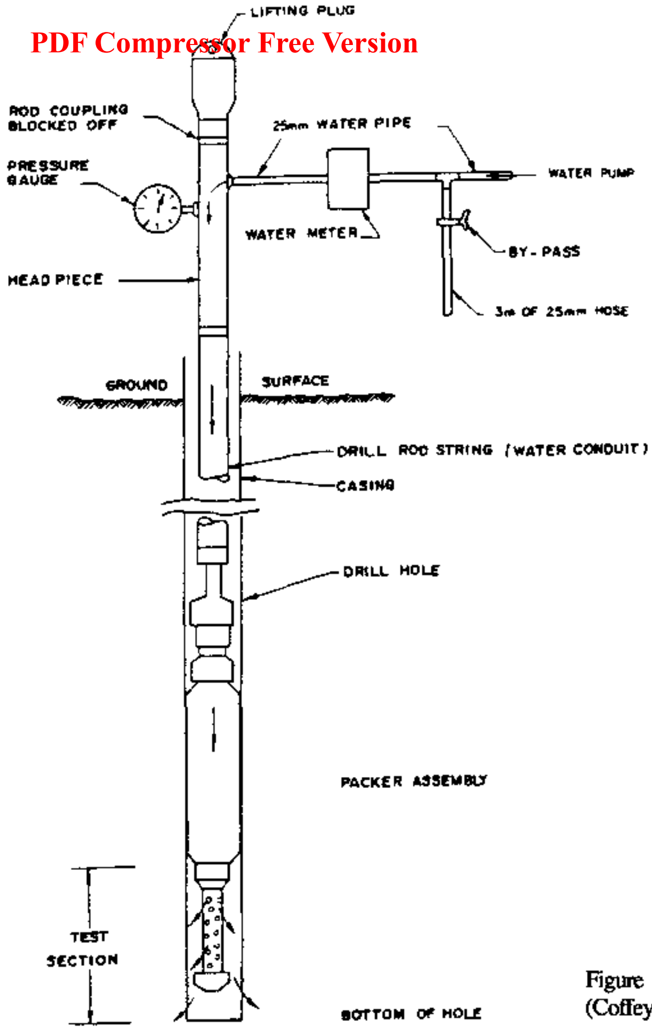


Figure 5.23. Hydraulic packer equipment (Coffey Partners International).

Hydraulic packers have proved reliable under most conditions. It is necessary to ensure that the hole outside the water line is kept full to indicate possible leakage past the packer and also to equalise the pressures and allow effective inflation and deflation of the sleeve.

– In a pneumatic packer the sleeve is inflated by air from a compressed air bottle using a separate air line to the surface. This method is effective in shallow holes, but with deeper holes fracturing of the air line has caused problems. Water levels do not affect inflation or deflation.

– Wireline packers have been developed for testing holes drilled with wireline equipment. These enable water pressure testing without the withdrawal of the drill rods. The packer incorporates two sealing sleeves, the upper seal within the drill rods and the lower seal in the hole below the drill bit.

– The mechanical packer seals by the expansion of two or more rubber rings when compressed (Fig. 5.24). The test section length is controlled by the insertion of a selected length of

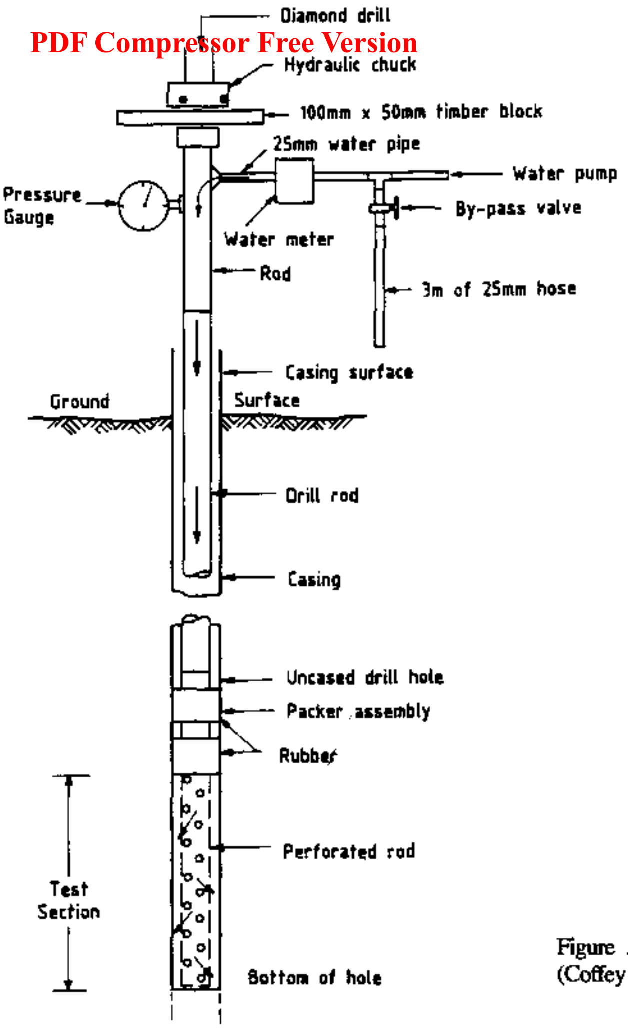


Figure 5.24. Mechanical packer equipment (Coffey Partners International).

perforated rod below the packer. Sealing is achieved by downward pressure on the drill string by the drill chuck. This pressure must be maintained throughout the test.

The capability of the rubber rings to expand is limited. With a sealing length of about 200 mm the formation of an effective seal in closely fractured rock is difficult. Alteration of the test section length involves removal of the whole drill string and the addition or removal of rods below the packer. This operation is time consuming in deep holes. Mechanical packers do however have the advantage of fewer operational problems.

– Double packers have two packers with a perforated connecting rod to form the test section. Problems with inflation may occur with both hydraulic and pneumatic packers. A double mechanical packer requires complete withdrawal and addition or removal of rods below the bottom packer between each test.

It is considered that mechanical packers should only be used where other packers are not available and results are not critical. Double packers should be avoided if at all possible.

5.12.4.2 *Water supply system*

The line connecting the packer to the surface should be watertight with minimum restriction to water flow. Flush coupled rods or casing are commonly used. The test water should be clean and supplied by a pump with bypass to enable control of pressure. Dirty water will result in clogging of fractures and lower Lugeon values than are correct for the rock.

A centrifugal pump or reciprocating pump with surge chamber is necessary to ensure constant pressure. Pressure and water flow is measured by meters which should be recently calibrated. Before testing the water supply, rods and packer, should be calibrated for friction losses at different flow rates.

5.12.4.3 *Selection of test pressures*

The objective of the test is to:

– measure the natural permeability,

– indicate the probable water flows under the expected hydraulic loading by the proposed structure.

The pressures applied during the test should not be sufficient to produce hydraulic fracturing of the rock around the test section. In weak rocks near the ground surface, this fracturing does take place at relatively low pressures and is usually indicated by a unexpected increase in water loss following a raising of test pressures. It is recommended that to avoid potential 'jacking,' maximum effective test pressures be limited below overburden pressure (approximately 22 kPa/m). In low strength weathered rocks, lower values will probably be necessary. The effective test pressure is the indicated pressure on the gauge plus the pressure exerted by the water column in the line between the gauge and the static groundwater level.

5.12.5 *Test Procedure*

The test involves:

– measurement of ground water level,

– washing out of drill hole. Circulation of drilling water should be continued for at least 15 minutes after the water appears clear,

– installation of packer at the selected level,

– connection of the water supply system,

– application of the test pressures and measurement of water loss,

– removal of equipment.

The testing is carried out in several stages with different pressures. Commonly at least three test pressures are used (five are desirable). Pressures are applied in an increasing and then decreasing sequence. For example with three pressures - a, b, c, - the water loss is measured at stages with pressure successively at a, b, c, b, a. Each stage should be continued until a constant rate of water loss (within 10%) for a 5 minute period is recorded.

5.12.6 *Presentation and interpretation of results*

The results are best plotted with the effective test pressure at the centre of the test section against the flow rate. The effective test pressure is the gauge pressure corrected for the elevation difference between gauge and water table, and for friction losses in the system. The flow rate is

HOLE NO.: JB24
TEST SECTION : 20.05m TO 26.50m
SECTION LENGTH : 6.45m

DURATION OF TEST mins	GAUGE PRESSURE kPa	WATER LOSS litres	litres per min	litres per min/m
5	20	20.0	4.0	0.62
5	20	19.0	3.8	0.59
5	60	28.5	5.7	0.89
5	60	26.5	5.3	0.82
5	110	36.0	7.2	1.12
5	110	35.0	7.0	1.08
5	170	48.5	9.7	1.50
5	170	46.5	9.3	1.44
5	110	34.0	6.8	1.05
5	110	32.0	6.4	0.99
5	60	25.0	5.0	0.78
5	60	25.5	5.1	0.79
5	20	15.5	3.1	0.48
5	20	16.0	3.2	0.50

INTERPRETED RESULT = 6 lugeons

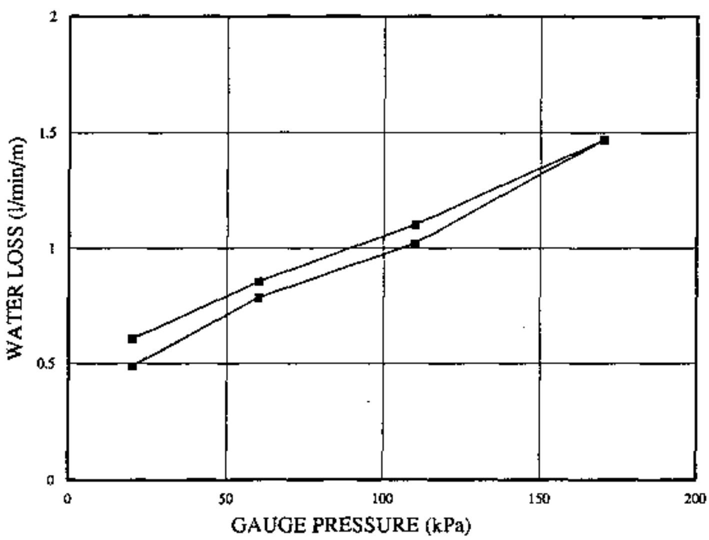


Figure 5.25. Typical results of packer test.

usually expressed in litres per minute per metre of test section length.

A typical test result is shown in Figure 5.25 where the data from the increasing pressures (a, b, c) do not plot identically to those on the decreasing pressures.

A range of different results is possible, indicative of the following possible mechanisms:

– Laminar flow and no change in permeability during the test.

– The occurrence of turbulent rather than laminar flow.

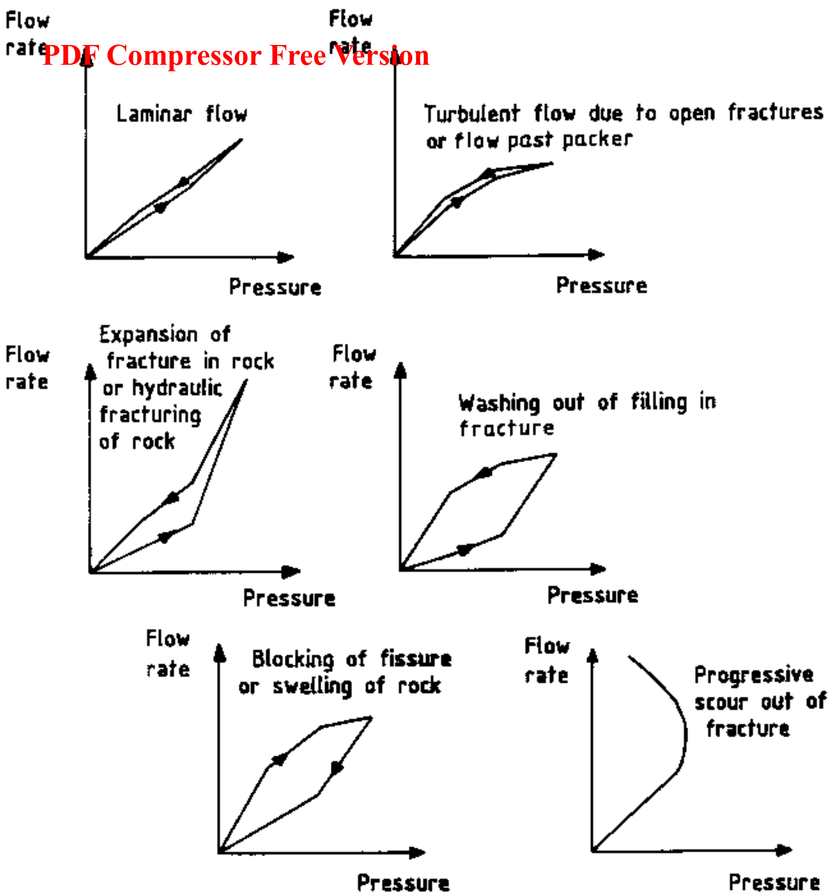


Figure 5.26. Typical pressure versus flow curves for packer tests.

– Scour of joint infill, weathered or crushed rock.
– Leakage past the packer.
– Sealing of joints by fines from the water.
– Hydraulic fracture of the ground.
– Inaccuracies in measurement.

Figure 5.26 shows examples of plots indicating these types of behaviour.

From the plotted data as shown in Figure 5.25 it is possible to judge which flow mechanism has occurred and hence to adopt the appropriate Lugeon value.

5.13 COMMON ERRORS AND DEFICIENCIES IN GEOTECHNICAL INVESTIGATION

Many problems during design and construction of dam structures are caused by poor quality or inadequate geotechnical investigations. This is often due to the investigation not following good engineering practice. Many of the problems are known to competent practitioners. In some situations lack of available finance can force the adoption of poor investigation practice. In many cases these inadequacies only become apparent during construction and lead to costly redesign and/or contractual disputes. Tables 5.4 to 5.8 list specific problems that occur in geotechnical investigations, the consequences, and measures which can be taken to avoid the problems.

Table 5.4. Deficiencies in drilling and sampling.

Problem	Consequences	Remedy
Only drilling vertical holes where jointing is near vertical	Joint spacing and rock mass permeability incorrectly assessed	Angle holes to intersect joints, bedding and other features
Poor identification of orientation of joints in boreholes	Lack of knowledge of actual joint orientation affecting slope stability	Careful orientation in boreholes and mapping in trenches
Water table not measured, not measured often enough or data not recorded	Dewatering problems with resultant contractual claims; lack of information for design, e.g. pore pressures for slope stability	Routine measurement and or recording. Install casing and or piezometers
Drill water inflows or outflows not recorded	Poorer understanding of reasons for high/low water tables	Employ good drillers and supervise full time
Poor identification of layering in soil deposits particularly when using augers below water table, or wash boring without sampling, e.g. see Figure 5.12 which compares log from augered borehole and static cone penetration test	Soil strata are mixed together, hole may collapse causing greater mixing. Leads to under/overestimation of horizontal/vertical permeability, potential overestimation of strength by mixing clay layers	Use rotary drilling with mud and/or casing, sample systematically with thin wall tubes and SPT. Use static cone penetrometers
'Blowing' in boreholes in silty sand, sand and soft clay i.e. flow of material towards borehole	Low SPT values in silt and sand and disturbed samples (in clay) leads to overestimation of settlement, underestimation of strength	Use drilling mud and excess head of mud in the borehole to prevent blowing
Only drilling holes, not test pits in 'structured' clays, e.g. fissured or lateritised soils, soils with root holes	Failure to recognise the structure usually leads to overestimation of strength and underestimation of permeability (often by orders of magnitude)	Dig backhoe and excavator pits and have experienced personnel log them
Drilling in gravel and gravelly sands with percussion drill	Gravel is broken up to finer particles, mixed with sand to give the impression of uniform sandy gravel. Fines may be lost if stratification is not recognised. Horizontal permeability is underestimated, possibly by orders of magnitude (see Fig. 6.32)	Recognise the problem. No real drilling solution. Test permeability with pumpout tests
Ground surface level and location of boreholes not surveyed	Errors in plotting sections, plans, misinterpretation of conditions	Survey all investations

Table 5.5. Deficiencies in in-situ testing.

Problem	Consequences	Remedy
Testing only at predetermined depths, e.g. 1.5 m, 3 m, 4.5 m etc., e.g. SPT, undisturbed tubing sampling	Poor identification of strata, poor selection of strata to test and sample	Supervise full time and test at strata changes as well as at predetermined depths
SPT (and CPT) test in gravelly soils affected by coarse particles	Overestimation of SPT 'N' value, with resultant overestimation of relative density, underestimation of compressibility	No real remedy. Just recognise the problem or seek other ways of estimating the parameters
Not washing borehole carefully before water pressure testing	Joints remain clogged with drill cuttings, lugeon value underestimated	Take care in washing hole. Use clean water for testing
Lack of in-situ permeability tests in soils	Contractual claims because 'conditions are worse than contractor assumed'; gross errors in estimation of permeability	Do appropriate in-situ tests despite the costs involved
Estimation of permeability from particle size distribution	Underestimation of permeability because of mixing of finer layers. See Figure 6.32	Only use 'Hazen' type formulae in uniform, clean fine-medium sand for which it was derived
Use of pump-in permeability tests in soil, particularly structured clay and in augered boreholes	Gross underestimation of permeability due to smearing and clogging of fissures, root holes, sandy layers, by factor of 10 to 10^3	Use pump-out tests where soils are below water table. Above water table use pits, with the sides carefully cleaned to remove smearing. Adopt 'realistic' values for design test results regardless of results
Use of seismic refraction survey to estimate rippability	Incorrect prediction of rippability, contractual claims	Do seismic refraction correctly (see Whiteley 1988). Couple with geological factors (see Braybrooke (1988), Fell (1988) and Stapledon (1988)), and recognise estimates are approximate in contractual arrangements
Installation of 'wells' instead of properly constructed piezometers in boreholes	Measures phreatic surface, not pore pressure. May over/underestimate pore pressures (see Fig. 10.3 and Fell 1987)	Install piezometers, properly sealed in borehole

Table 5.6. Deficiencies in logging of boreholes and test trenches, pits.

Problem	Consequences	Remedy
Logging rock in soil description terms, e.g. 'black silty clay' in holes drilled by auger, with the rock ground to soil to consistency by the drilling bit	Contractual claims relating to difficulty of excavation; incorrect design assumptions	Use correct drilling techniques and/or log correctly and/or log in soil and rock terms
Logging joints, partings and drill breaks all together as 'fractures'	Incorrect assessment of the joint spacing leading to incorrect assessment of ease of rippability and size of ripped rock and contractual claims; incorrect assessment of slope stability and grouting conditions	Log joints, bedding plane partings and drill breaks separately and present data clearly
Failure to log condition of joints, e.g. clay coating, iron stained, and continuity of joints. Poor definition of weathering classification, particularly highly weathered vs moderately weathered	Incorrect assessment of joint strengths in slope stability, and of likely flowrates of water, grouting conditions, rippability. Confusion on acceptable rock conditions during construction with resultant contractual claims	Use experienced personnel to log core (or at least check logging). Define weathering classification and/or define acceptable conditions accurately
Incorrect description of cemented soils, either failing to describe the cementing, e.g. in calcareous sands OR describing cemented soil as rock	Incorrect assessment of conditions for design and contractual claims for excavation or tunnel support conditions	Inspect exposures in large cuttings, dig pits, related drilling results to the local geology and log accordingly, (e.g. Stapledon (1988))
Failure to log soil structure, e.g. fissures, root holes, minor interbedding	Incorrect assessment of shear strength of fissured soils, underestimation of permeability	Log carefully and systematically in pits
Classifying soils in the dry state	Underestimation of clay content and plasticity leading to incorrect design specification and contractual claims	Moisten soil before classifying
Incomplete description of soil, e.g. omission of moisture conditions, consistency, colour	Incorrect assessment of conditions for design, specification and contractual claims based on unforseen 'wet soil' etc.	Log carefully and systematically
Inadequate description of 'organic matter' and 'fill'	Contractual claims and incorrect design	Describe in detail in the logs

Table 5.7. Deficiencies in data presentation and interpretation.

Problem	Consequences	Remedy
Too much detail on irrelevant features, e.g. lengthy description of trench logs, or mineralogy of rock types in borelogs. Confusion of 'facts' and interpretation	Important features lost in the mass of irrelevant data. Misinterpretation of geological conditions, potential for contractual claims	Proper planning and briefing of personnel by experienced geotechnical practitioners. Clear distinction and definition of terms
Straight line interpolation between boreholes without regard for lack of data or geological conditions	Misinterpretation of geological conditions, overconfidence in interpolation, potential for contractual claim	Draw interpretive sections with due allowance for geology e.g. core stones in granite, buried land surfaces
Use of exaggerated scales in preparing sections	Misinterpretation of geological conditions, overconfidence in interpolation, potential for contractual claim	Use natural scale whenever possible or provide both natural and exaggerated scale
Consideration of data on a hole to hole basis	Misinterpretation of geological conditions, incorrect assessment of design parameters and the range of values	Determine a proper geotechnical model of the site based on all data
Failure to recognise that it is the exception which sometimes causes the problems e.g. a thick bed of rock or a thin bed of high strength rock causes problems with rippability and size of ripped material	Contractual claims	Recognise the importance and include in reports and interpretive sections
Incorrect projection of information from boreholes, in particular altering levels	Incorrect interrelation of conditions	Project along contour and/or with consideration of geological controls

Table 5.8. Deficiencies in earthfill borrow area investigations.

Problem	Consequences	Remedy
Use of boreholes to investigate earthfill and sand/gravel deposits	Failure to recognise variability of soils, water contents incorrectly determined, contractual claims	Use backhoe or excavator pits where possible (< 6 m)
Lack of water content profiles, or profiles taken in periods not typical of contract	Lack of knowledge of moisture condition requirements, contractual claims	Take profiles in period representative of construction
Lack of information on 'bulking' factors in earthworks	Deficiency of material when 'shrinkage' occurs leading to contractual claims	Do density in place tests and laboratory compactions in borrow area investigations
Variability of alluvial and non alluvial soil deposits	Difficulty in selecting materials during construction, contractual claims	Dig plenty of test pits

CHAPTER 6

Laboratory testing techniques and their limitations

6.1 SHEAR STRENGTH OF SOILS

When designing earthfill, or earth and rockfill dams and/or dams on soil foundations, it is necessary to carry out laboratory tests on samples of the soil to determine the shear strength for use in stability analysis. In practice the majority of laboratory testing is carried out on cohesive soils, i.e. clay, sandy clay and clayey sand. It is difficult to obtain undisturbed samples of cohesionless soils i.e. gravel, sand, silty sand and silt for testing. If these soils are present in the foundation it would be normal to estimate their shear strength (i.e. effective friction angle) from *in situ* tests (see Chapter 5). Some laboratory testing may be carried out on cohesionless foundation soils compacted to the same relative density as estimated in the field from the *in situ* tests. Tests can also be carried out on cohesionless soils used for the embankment, e.g. for filters, although in many cases these properties will be estimated as they are usually not the weakest zone and therefore not critical for stability analyses of dams.

Historically both total and effective stress methods of stability analysis have been used (Sherard et al. 1973). However, the authors' strong recommendation is that only effective stress analyses be carried out for assessing the stability of the embankment under static loading (i.e. with no earthquake effect). The effective stress strength parameters c', ϕ' can be estimated with some degree of confidence provided the tests are carried out correctly, and in general pore pressures can also be estimated reasonably accurately (or conservatively) allowing a confident assessment of the factor of safety. Where pore pressures are critical and cannot be estimated accurately they can be monitored by piezometers.

If total strength parameters are used for design, e.g. for earthfill, their assessment is totally dependent on estimated water contents, density of compaction, earth pressures in the dam and pore pressure dissipation. Since these are seldom known accurately, and the strength is very sensitive to variation in these assumptions, it is not possible to predict factors of safety with any degree of confidence. Unlike effective stress analyses where the major variable, the pore pressure, can be readily monitored, the undrained strength cannot be measured *in situ* with any degree of confidence.

The following discussion on shear strength of soils concentrates on testing of cohesive soils to determine the effective strength. It is the authors' experience that many organisations, even large and reputable ones, do not follow closely the correct testing procedures, despite the fact that these have been long established. The emphasis in the discussion, therefore, is on the problems which arise in testing and ways to avoid them. This discussion is substantially similar to that presented by Fell & Jeffery (1987) and the contribution of R.P. Jeffery is acknowledged.

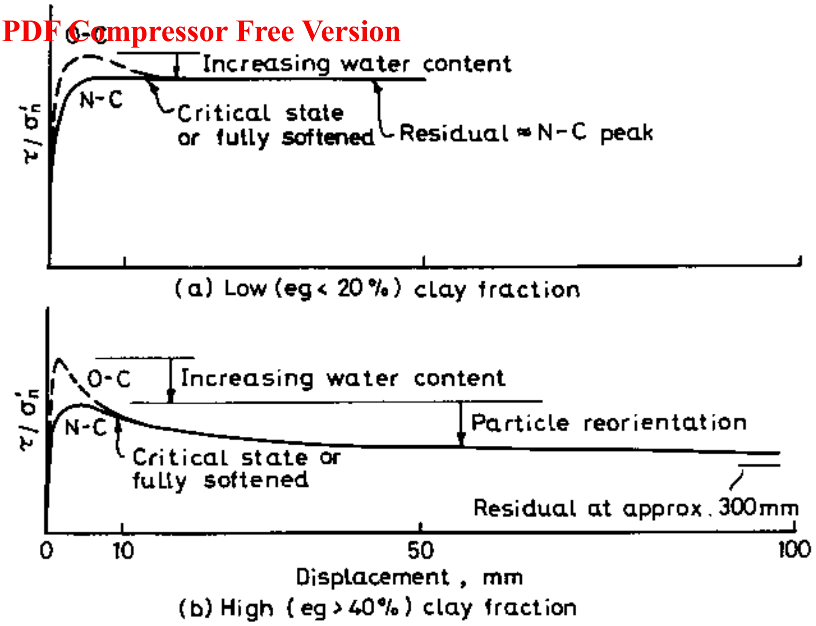


Figure 6.1. Diagrammatic stress displacement curves at constant normal stress ϕ'_n (Skempton 1985).

6.1.1 *'Drained' or 'effective' shear strength parameters – Definitions*

When a soil is sheared slowly in a drained condition (so that there is sufficient time for dissipation of pore pressures induced by shearing), the stress displacement curves will take the general form shown in Figure 6.1.

The behaviour is dependent on whether the soil has a high or low clay fraction (finer than 0.002 mm) content and on whether it is normally consolidated (NC) or overconsolidated (OC), but has the following common features:

– A 'peak' strength is obtained at a small displacement.

– A reduction of strength to the critical state or fully softened strength then occurs with further displacement. For overconsolidated soils this is due to increase in water content with dilation of the soil as it is sheared. The fully softened strength corresponds to the critical state (Skempton 1985) i.e. when continuing displacement occurs without further change in volume or water content. Note that normally and overconsolidated samples of the same soil will tend to achieve the same critical state or fully softened condition.

– With continuing displacement of soils with a high clay fraction content, particle reorientation occurs, resulting in a further reduction in shear strength. The minimum value of shear strength achieved at large displacements is the 'residual' strength.

– The residual and fully softened strengths are significantly different for high clay fraction content soils, but not for soils with low clay fraction content (see Figure 6.1). Sands behave similarly to the low clay fraction soils.

Strength envelopes for these cases are defined by:

– peak strength: c' ϕ' (or less com monly c'_p ϕ'_p).

– softened strength: c'_S ϕ'_S.

– residual strength: c'_R ϕ'_R.

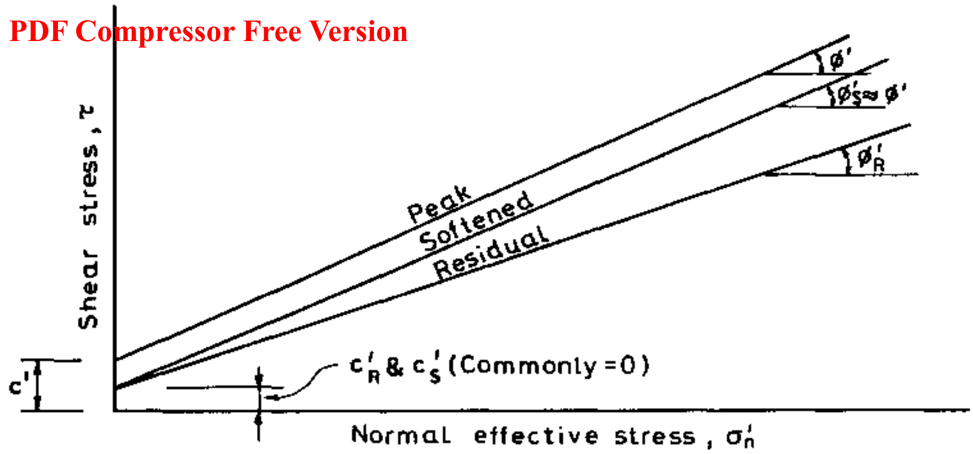
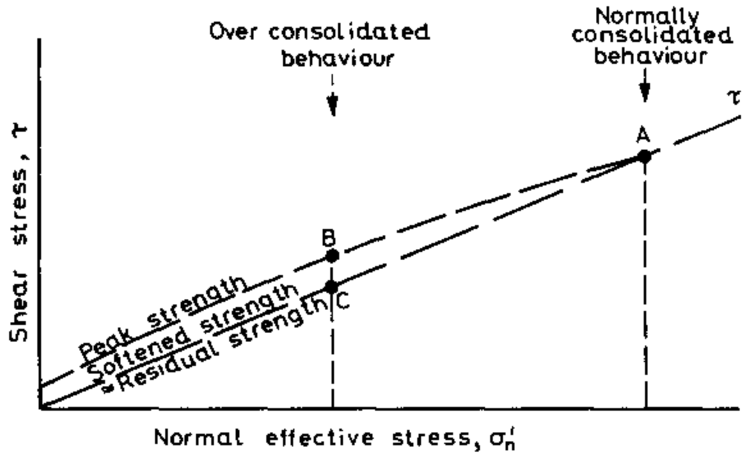


Figure 6.2. Relationship between peak, softened and residual strengths.



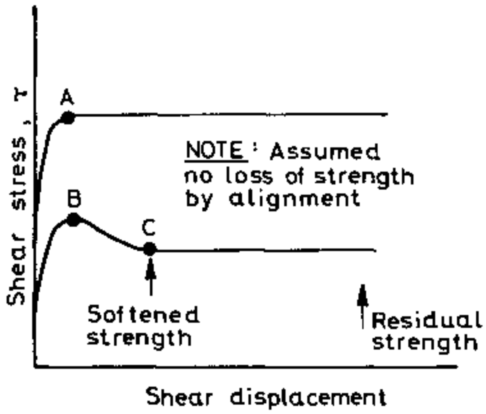


Figure 6.3. Diagrammatic stress displacement behaviour.

Note that often $c'_S \approx c'_R \approx 0$, and $\phi'_S \approx \phi'$.

Figure 6.2 shows the relationship between these parameters.

Skempton (1985) adopts the use of the term 'field residual' strength (ϕ'_{RF}) as the strength of fully developed shear or slide surfaces in nature. As discussed in Section 6.1.3 this value differs from the laboratory residual strength depending on the testing method.

In practice, it may also reflect the roughness and waviness of the field failure surface compared to the planar surface in the laboratory sample.

An undisturbed sample of soil may behave in an overconsolidated manner at low normal stress and in a normally consolidated manner at high normal stresses (in excess of the preconsolidation pressure). This affects the pore pressure response of the soil during shear and also the load-deformation behaviour as shown in Figure 6.3.

6.1.2 *Development of residual strength*

There is a large amount of evidence that softening, with increased water content, and particle reorientation, occurs on slide planes and that these lead to a reduction in shear strength from the peak strength. The papers by Skempton (1985), Lupini et al. (1981), Mesri & Cepeda-Diaz (1986) and Hawkins & Privett (1985) give good summaries of these effects.

Lupini et al. (1981) carried out a series of ring shear tests on sand-bentonite mixtures, and suggested that there were different mechanisms of residual plane development depending on the clay fraction percentage present. The mechanisms are turbulent or rolling shear, where the presence of rounded silt size particles prevents alignment of clay particles on the shear surface; sliding shear when the effects of the silt is overridden by the predominance of clay particles; and a transition between these two conditions. These results were summarized by Skempton (1985) and are reproduced in Figure 6.4.

Skempton (1985) presented results of field residual and ring shear tests on a range of soils. These are reproduced in Figure 6.5.

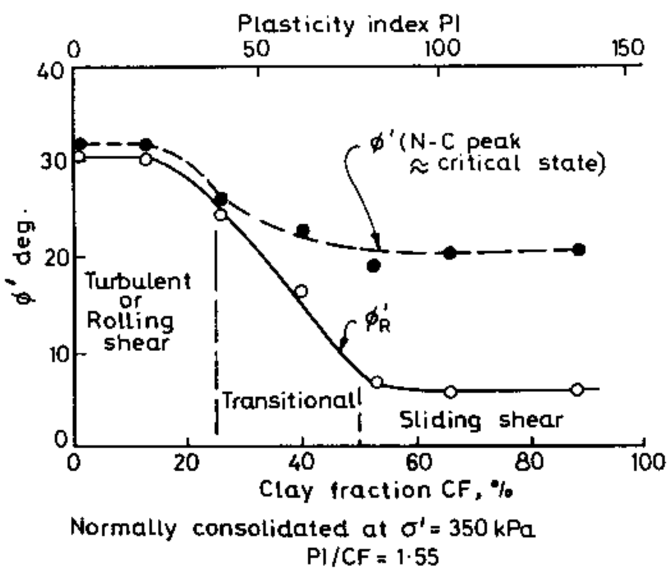


Figure 6.4. Ring shear tests on sand-bentonite mixtures (from Skempton 1985 after Lupini et al. 1981).

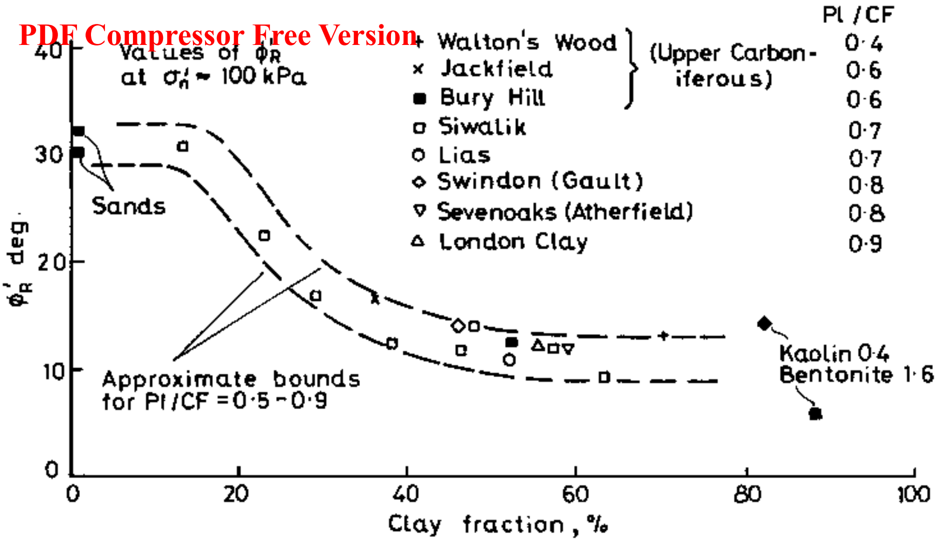


Figure 6.5. Field residual and ring shear tests on sands, Kaolin and bentonite (Skempton 1985).

These figures show that 'sliding shear' with complete reorientation of the clay particles is likely to occur only where the clay fraction (finer than .002 mm) exceeds 50% of the total soil, and that for less than about 25% clay fraction, 'turbulent' or 'rolling' shear occurs without the influence of clay particle alignment. Note that Figure 6.5 only applies to clays with a PI/CF ratio of 0.5 to 0.9.

Apart from the clay fraction, the mineralogy of the clay also has an effect on residual strength. This is particularly so when the clay fraction is large. This reflects the fact that the different clay minerals have different particle shape and different interparticle bonding. Most clay minerals, e.g. kaolinite, illite, chlorite and montmorillonite are platey structures, and are therefore subject to alignment when sheared. Montmorillonite has a particularly thin plate structure, and weak interplate bonds and leads to the lowest residual strength values of ϕ_R' as low as 5°. Kaolin has a ϕ_R' ÷ 15° and illite approximately 10°.

Some clay minerals do not have a plate structure, e.g. halloysite has a tubular structure, attapulgite a needlelike structure, and some amorphous clay minerals such as gibbsite, haematite, bauxite have essentially granular structures. This leads to much higher residual friction angles, commonly greater than 25° (Skempton 1985).

6.1.3 *Selection of strength for design*

6.1.3.1 *Peak, residual or fully softened strength*

Whether peak, softened or residual strengths are used in the analysis of slope stability depends on the presence or absence of existing slide 'planes' (actually surfaces which may not be truly planar) and fissuring. The following guidelines are given:

– Where there is an existing slide plane, e.g. in the foundation of a dam, or where a dam has failed by sliding, the field residual strength should be used for the slide plane. This applies regardless of how long it has been since sliding last occurred.

– The exception could be where there is definite evidence of recementing, e.g. in some slickensided joints in weathered old rocks.

– Bedding plane shears formed by folding of rock strata have strengths approaching residual, and unless extensive tests on the planes show otherwise, residual strengths should be used.

– Compacted soils, and soils which have no fissuring should be assigned peak strength parameters c′ and ϕ′.. The majority of dam embankment design is therefore based on peak strength parameters.

– Fissured soils have a strength between peak and residual strength depending on the nature of fissuring, orientation, continuity and spacing of the fissures. Such soils do occasionally occur in dam foundations, e.g. Ross River Dam (McConnel 1987). This is discussed in more detail in Section 6.1.5. Soils with relic joints, such as extremely weathered basalt, behave in a similar manner.

– Triaxial and direct shear tests do not properly simulate the actual plane strain stress conditions which exist in most slope stability problems. However, the uncertainty arising from this is not significant in practical slope stability problems, and provided the tests are carried out and interpreted correctly the results from both tests can be adopted for design.

It should be noted that as pointed out by Lade (1986) and Mitchell (1976), apart from cemented soils, the effective cohesion (c′) should be zero or very small. The value of c′ adopted is particularly critical in design of smaller dams and in landslide slope stability work, because it has a major effect on calculated factors of safety when failure surfaces are shallow. It is recommended that unless there is definite evidence of higher values, the effective cohesion adopted for design should be between 0 and 10 kPa for peak strength and should be zero or say 1 kPa for residual and softened strength, at least at low normal stresses.

6.1.3.2 *Selection of design parameters*

When several triaxial tests have been carried out on the one soil, it is recommended that the design shear strength parameters are obtained from a p-q plot of the test results, rather than by say averaging the individual c′, ϕ′ values from each test, or plotting all Mohr's circles on one diagram. It is also important to use results from the effective stress range applicable to the field problem – in many cases this is at low stress, e.g. less than 50 to 100 kPa for slide surfaces at 5 m depth.

The p-q plot is a graph of the apex points of the Mohr's circles from the test results as shown in Figure 6.6

When selecting the design parameters for design of dam embankments and landslide stabilizing works, it is common to bias towards the conservative by selecting a line with say 75% of the test points above, 25% below (i.e. a lower quartile line), but a line of best fit, or a lower bound may be adopted depending on the circumstances.

If direct shear tests have been used to determine strength parameters the results should be plotted on a graph of normal stress vs shear stress and design parameters selected by a similar procedure to that outlined below for triaxial tests.

In any slope stability analysis it is good practice to check the calculated factor of safety for a range of strengths, e.g. lower quartile and lower bound, to determine the sensitivity of the factor of safety to the assumed strength.

It is essential to use judgement in selecting the design line, rather than using say a least squares regression analysis, as judgement enables allowance to be made for:

– poor individual test results,

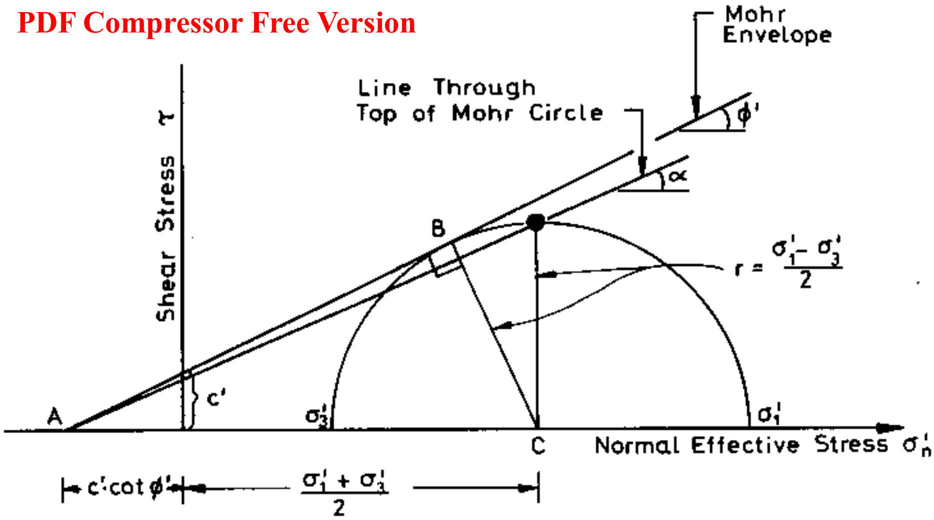


Figure 6.6. Basis of p-q plot where $p = (\sigma_1' + \sigma_3')/2$; $q = (\sigma_1' - \sigma_3')/2$; it can be shown that $\phi' = \sin^{-1}(\operatorname{Tan}\alpha)$; $c' = a/\cos\phi'$ where α and a are obtained from the p-q plot as shown on Figure 6.7.

– the general trend for the second and particularly third stage of staged tests to give a lower strength than the true strength because of excessive deformation, and some loss of strength from the peak due to displacement on the shear plane, particularly in sensitive clays or cemented soils;

– general curvature of the Mohr's circle envelope,

– adoption of low effective cohesion (c') which correctly models the behaviour of most soils. The average of the c' and ϕ' values from which the p-q plot was derived, is also shown in Figure 6.7. It can be seen that using averages tends to give a larger c' and lower ϕ', than using the p-q diagram. This is generally unconservative for smaller dams and landslide stability because the strength is overestimated in the working stress range.

Wroth & Houlsby (1985) have suggested a method based on the critical state parameters. This method is similar in principle to that proposed above using a p-q diagram, but takes account of the variation of c' with water content at failure. The critical state P, Q values [$P = (\sigma_1' + 2\sigma_3')/3$ and $Q = (\sigma_1' - \sigma_3')$] at failure are normalised by dividing by the equivalent pressure at that water content (the pressure on the normally consolidated line at the same voids ratio as the specimen). Wroth and Houlsby claim that this gives a more rational way of selecting c' as a function of water content, particularly for fills. In practice this degree of sophistication may not be warranted.

Handy (1981) suggests that it is incorrect to use linear regression curve fitting to p-q plots of triaxial data, where the variation in q is considered, but no variation in p assumed non variable. He suggests that the direction of variability is at about 45° to the p-q axes and suggests rotating the axes 45°, carrying out a linear regression analysis on the rotated axes, then reconverting both to the original axes. Handy claims this introduces bias on the safe side. However for the example given in Handy, the method yielded a large cohesion intercept, and would most likely have overestimated the shear strength at low normal pressures. Because of this and because it does not facilitate judgement, as outlined above, the method is not recommended.

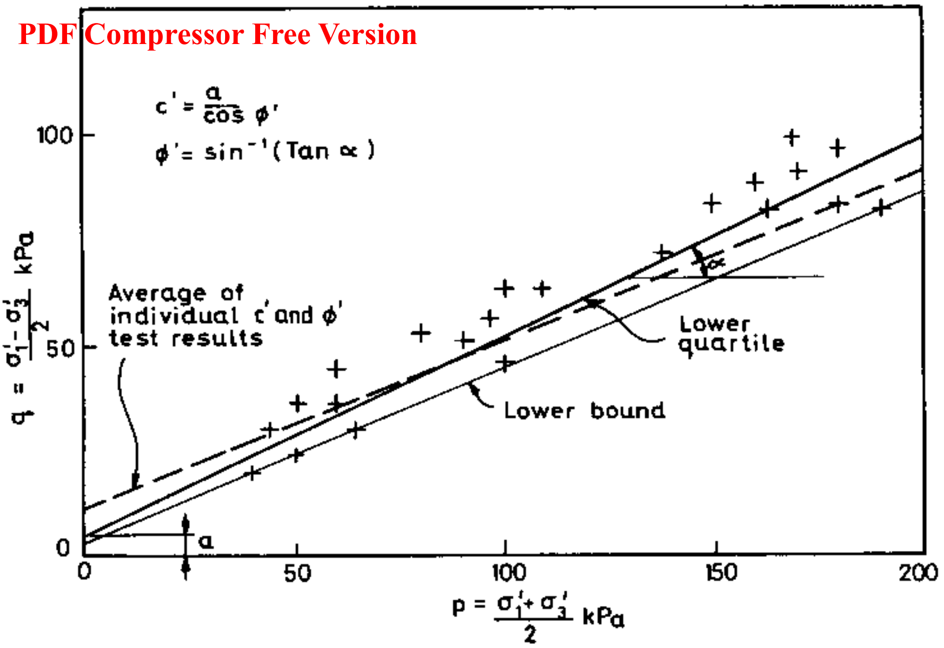


Figure 6.7. Typical p-q plot of triaxial test results.

6.1.4 *Triaxial shear test procedures and common errors*

6.1.4.1 *Test procedures and application*

Triaxial tests are the most common means of obtaining the peak shear strength parameters c′, φ′. In general the triaxial test cannot be used to obtain softened or particularly residual strengths because it is not possible to subject the soil to sufficient displacement on the shear surface to reach these values.

Most triaxial testing is triaxial compression, carried out at a nominally constant rate of axial strain. Tests are usually saturated consolidated undrained with pore pressure measurement (CUDPP) or saturated consolidated drained (CD). For practical purposes these yield the same strength provided the tests are performed correctly. Most triaxial testing is CUDPP because the time for testing is less than for CD. However, in testing some soils, the pore pressure response during testing is such that the CUDPP circles fall close together, and CD tests may be needed to obtain a wide enough range of Mohr circle plots to draw an accurate envelope. Staged testing, where one sample is tested for three consolidating stresses is sometimes used because it involves less sample preparation, but can lead to errors as described below.

Details of testing equipment and test procedures are given in Head (1985), Bowles (1978) and Bishop & Henkel (1971). Lade (1986) and Saada & Townsend (1981) give good summaries of the current state-of-the-art of triaxial testing.

6.1.4.2 *Common sources of error in triaxial testing*

There are several common sources of error in triaxial testing, which often lead to an overestimation of effective cohesion with a reduction of effective friction angle. This may lead to an

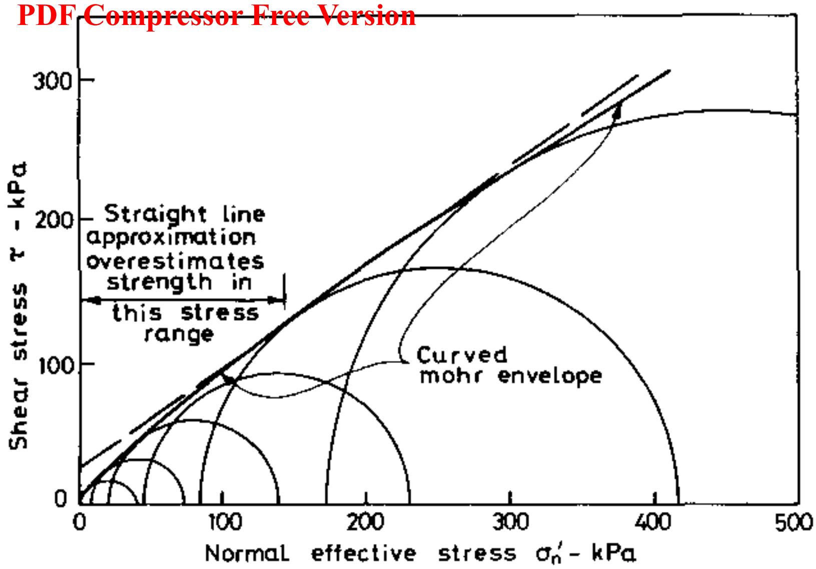


Figure 6.8. Typical Mohr's envelope plot.

overestimation of strength in the working stress range.

(i) Testing at a too high stress range. The Mohr's circle envelope for most soils is curved in the manner shown in Figure 6.8. If laboratory tests are carried out at higher stresses than will be encountered in the field, this will result in an overestimation of shear strength in the field working stress range – for Figure 6.8 if the field stresses were between 0 and 100 kPa and testing had been carried out confining stresses between 100 and 400 kPa, the strengths would be overestimated by 10 to 300% depending on the actual normal stress. This potential problem can be overcome by specifying the correct stress range for the triaxial tests and recognising possible curvature effects when selecting design parameters from the p-q plot. It is also important to plot the individual Mohr's circle plots for a sample, so the curvature can be identified.

(ii) Not saturating the sample adequately. Soil samples (even if taken from below the water table) may not be saturated before testing in the laboratory because:

– fissures in the soil open up due to sampling and unloading,
– pore pressure redistributes in non homogeneous soils on unloading,
– air intrusion may occur at the surface of the sample,
– dissolved gas may come out of solution,
– air may be trapped between the sample and the membrane in the triaxial test,
– compacted soils even if compacted wet of optimum water content as is commonly required in dam construction are usually only 90 to 95% saturated.

The effects of partial saturation are:

– pore pressure changes during shearing are less than for a saturated sample. However, the trend (whether +ve or -ve changes) is not affected (Lade 1986).

– because pore pressure changes are less than for a saturated soil, the undrained strength will be markedly affected: For overconsolidated clays, Δu is -ve, and lower than for a saturated soil, so the undrained strength is reduced; for normally consolidated clays, Δu is +ve and lower than for a saturated soil, so the undrained strength is increased.

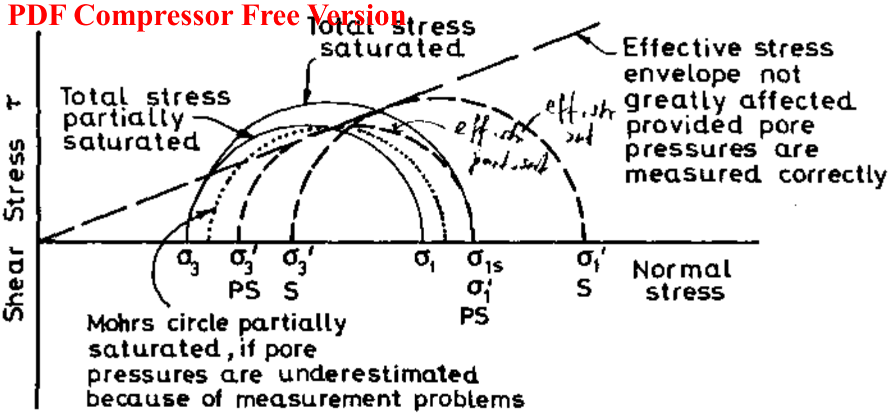


Figure 6.9. Effects of partial saturation of triaxial testing. S = saturated sample; PS = partially saturated sample.

– the magnitude of actually measured pore pressures may be incorrect, usually underestimated, because of the presence of air in the pore system.

Provided the pore pressures are measured correctly, the effective strength envelope will not be greatly affected even though the total stress envelope is markedly affected.

Figure 6.9 shows these effects for an overconsolidated clay.

To avoid problems of partial saturation, the laboratory samples should be saturated by percolation followed by back pressure saturation. Percolation by itself will not achieve saturation even in permeable soils (e.g. sands). Back pressure saturation is needed to reduce the volume of the air bubbles in the pore water (Boyle's Law) and to drive air into solution (Henry's Law). The degree of saturation should be checked by monitoring the increase in pore pressure for an increment in cell pressure in undrained conditions:

$$\Delta u = B\,\Delta\sigma_3$$

Where B is defined by Skempton (1954) as

$$B = \frac{1}{1 + \dfrac{nC_v}{C_{sk}}}$$

where n = porosity

C_v = compressibility of pore fluid

C_{sk} = compressibility of the soil skeleton.

For a saturated soil $C_v = C_{water}$ and C_v/C_{sk} is very small, hence B = 1.

However, for very stiff soils, and very weak rocks, C_{sk} is also small and it is difficult to achieve B = 1 as shown in Figure 6.10.

In these circumstances it is sufficient to check B for several increments of cell pressure, and provided B is constant, the correct effective strength parameters will be obtained (Lade 1986).

For very stiff soils and very weak rocks, the stiffness of the testing system becomes significant and may lead to low measured B values.

Typically back pressure saturation will be carried out at 300 to 1000 kPa, but may be as low as 200 kPa for soft soils. It should be noted that dissolving the air in the water takes time (often

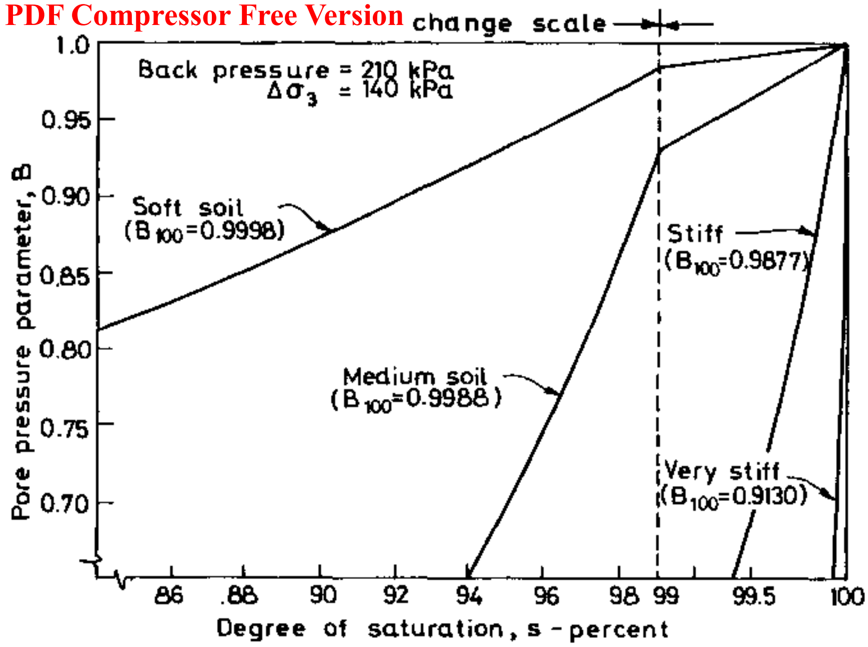


Figure 6.10. Variation of B-value with degree of saturation for four classes of soil (Black & Lee 1973).

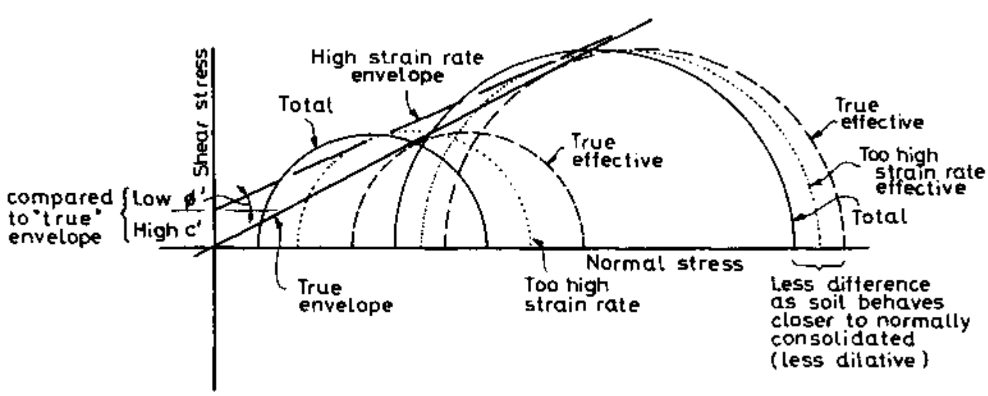


Figure 6.11. The effect of high strain rate on CUDPP triaxial tests.

many hours). Black & Lee (1973) give a method to calculate this time.

(iii) Testing at too high a strain rate. CD and CUDPP triaxial tests must be sheared at a sufficiently slow rate to allow:

– dissipation of pore pressure in the CD test,

– equalisation of pore pressure throughout the sample in the CUDPP test.

The effect of testing at too high a strain is illustrated in Figure 6.11 for an overconsolidated clay.

If a CUDPP test is sheared too quickly, the pore pressure changes measured at the ends of the sample are less than the actual changes at the centre of the sample where shearing is occurring.

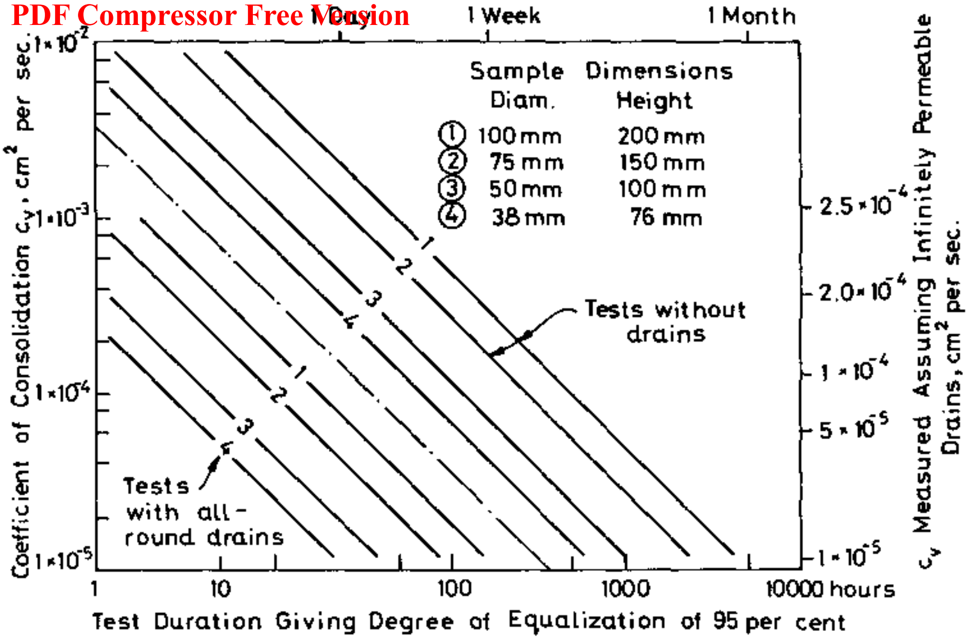


Figure 6.12. Chart for finding durations of drained and undrained tests for 95% dissipation at failure (Blight 1963 copyright ASTM.Reprinted with permission).

Since there is a reduction in pore pressure during shearing of an overconsolidated clay, the Mohr circle plots based on measured pore pressures are displaced to the left of where they should be.

The negative pore pressure response to shearing is less for the sample at higher confining stress where the soil behaves closer to a normally consolidated soil, so the displacement of the Mohrs circle for the higher confining stress is less than that for the lower confining stress.

This yields overall an increase in c′, and a decrease in ϕ'. The effect is similar for CD tests, where negative pore pressure will remain in the centre of the sample if the strain rate is too high, giving an apparent increase in failure stress (σ_1'), which again will be greater for samples at lower confining stresses than for higher confining stresses.

There are well established methods for determining a strain rate which will allow dissipation/equalisation of pore pressure. Bishop & Henkel (1971) suggest:

$$Tf = \frac{20H^2}{U\,c_v}$$

where Tf = time for failure

H = half height of sample

c_v = coefficient of consolidation

U = drainage boundary condition factor.

c_v is obtained by monitoring the consolidation phase of the test.

Blight (1963) presents a graph (Fig. 6.12) from which the test duration can be determined. This is based on 95% equalisation (dissipation) of pore pressure which is accepted as realistic.

Akroyd (1957,1975), Head (1985) and Bowles (1978) all give relationships for estimating time for failure based on monitoring the consolidation stage of the test.

It should be noted that c_v calculated from the first stage consolidation may overestimate c_v for the later stages and, hence, overestimate the strain rate required to achieve pore pressure equalisation or dissipation.

Lade (1986) indicates that by using lubricated end plates, pore pressures are more uniform and CUDPP tests can be carried out more quickly. However no guidelines are given to determine an acceptable strain rate. Lubricated end plates do not reduce the required time for failure in CD tests since pore pressure dissipation, not equalisation, is required.

(iv) Staged testing. In staged testing, the one soil sample is usually saturated, consolidated and sheared at the lowest selected confining stress, then consolidated to the second confining stress and sheared, then further consolidated to the third confining stress and sheared. This reduces the amount of sample preparation time, and hence the cost of testing. Where there are fabric features present (e.g. fissures), staged testing is desirable so the complete test is carried out on the one feature. The procedure generally gives acceptable results, but may tend to give lower strengths for the second and particularly third stage due to sample deformation, and loss of strength due to displacement on the shear plane.

On larger projects the third (and second) stage testing should be checked with tests on samples consolidated to the third stage confining pressure without earlier staged testing. Staged testing should not be used for sensitive or cemented soils.

(v) Laboratory preparation of compacted earthfill. The shear strength c',ϕ' of a compacted earthfill will be somewhat dependent on the degree of compaction with higher compaction, leading to minor increase in c' and ϕ'. When testing earthfill for dam construction, the laboratory tests should be carried out on samples compacted at the lower limit of the specified compaction, e.g. at 95% standard maximum dry density. The compaction water content should be at the upper limit of the specified water content (e.g. +1% of optimum water content) to facilitate saturation of the soil before testing.

6.1.5 *Triaxial testing of fissured soils*

The presence of fissures in soil reduces the mass shear strength compared to the soil substance strength. The strength of the fissured soil mass is dependent on:

– the nature of the soil substance,

– orientation, surface geometry, continuity, spacing and surface roughness of the fissures.

Where fissured soils are tested in the laboratory, these factors have to be considered in relation to the test sample.

Thorne (1984) summarizes the effects of fissuring on the shear strength. He draws on the papers by Skempton & Petley (1967), Lo (1970), Marsland (1971) and McGown et al. (1974) to conclude that:

a) The effective strength of fissure surfaces depends on the nature of the surface; dull fissures have $c'_f=0, \phi'_F=\phi'_{peak}$ (i.e. peak friction angle) for the soil without fissures (i.e. dull fissures have a strength approximating to the softened strength). Slickensided fissures have $c'_F=0, \phi'_F=\phi'_R$ where ϕ'_R = residual friction angle for the soil. Figure 6.13 shows this effect.

b) The size of the sample being tested has a major influence on shear strength. Figure 6.14 shows that the undrained strength may be overestimated by up to 3 to 7 times if small samples are used. The effective strength envelope will also be effected.

This can lead to a requirement for very large samples, e.g. 300 to 600 mm diameter for fissured clays Thorne (1984). Rowe (1972) suggests that for fissured soils a minimum speci-

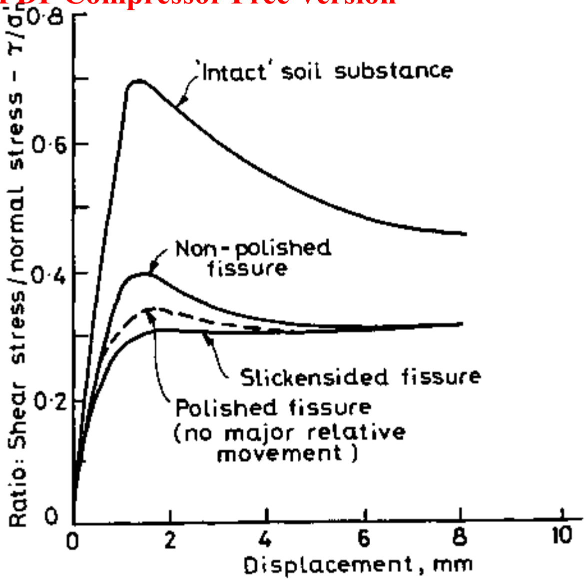


Figure 6.13. Effect of the nature of the fissure surface on shear strength (Thorne 1984).

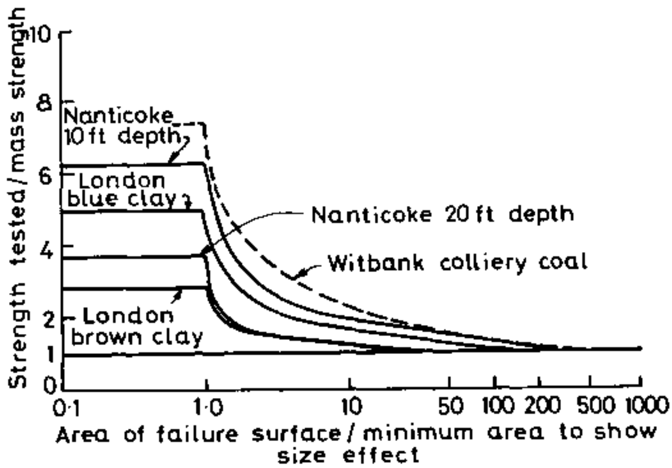


Figure 6.14. Effect of sample size on measured undrained strength (Thorne 1984).

men size should be 100 mm diameter, but this is dependent on fissure geometry, and samples up to 250 mm diameter may be required.

A practical approach for fissured soils which cannot be sampled by block sampling in open excavation which was adopted by Thorne (1984) was to:

– test 75 mm diameter samples, being the maximum size which can readily be obtained with thin wall samples tubes;

– carefully examine the samples before and after testing to determine whether shearing has occurred on a fissure surface or through the soil substance, and note the orientation of the failure surface;

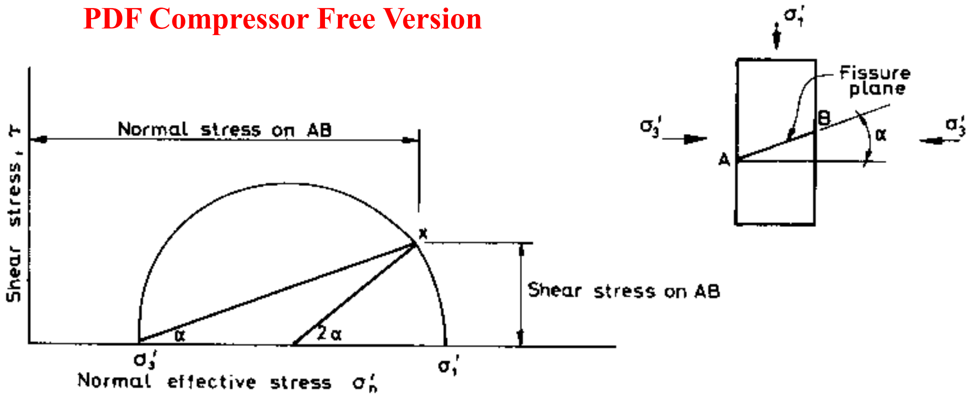


Figure 6.15. Resolution of stresses on failure surface.

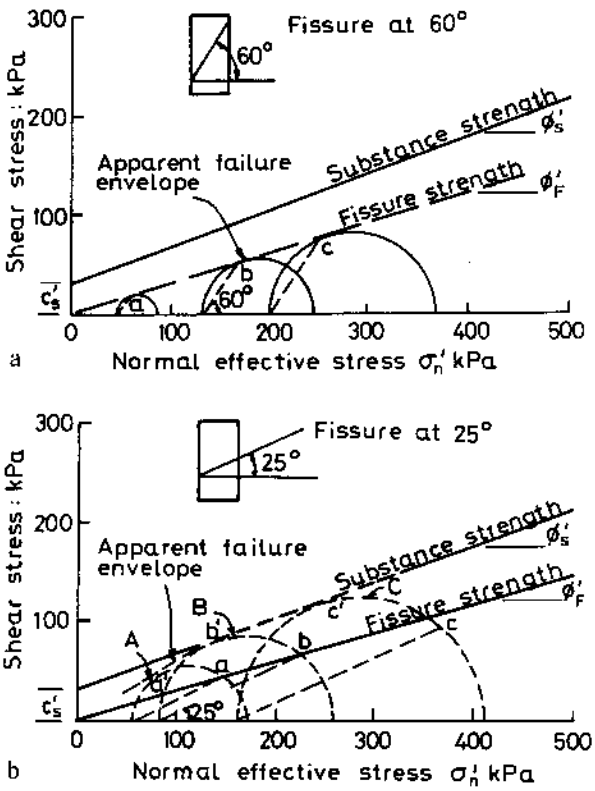


Figure 6.16. Mohr's circle envelopes for fissures at 60° and 25° (Thorne 1984).

– determine the shear stress on the fissure surface from the Mohr's circle using the construction shown on Figure 6.15;

– plot the strengths on fissure surfaces in this manner to determine the design fissure strength (as a p-q plot of points).

Thorne (1984) shows that whether a sample fails on a fissure depends on the orientation of the fissure and the magnitude of the stresses. Figure 6.16 illustrates this.

For Figure 6.16a, 60° fissure: A, B and C all fail on fissure.

For Figure 6.16b, 25° fissure: Circle A – fails on substance.

For fissured soils which can be sampled by block sampling from test pits, shafts or other excavations, it is often practical to test the actual fissure surface in a direct shear test to obtain the fissure strength.

Having arrived at a fissure strength, it is necessary to determine the overall soil mass strength.

If fissures are randomly oriented and continuous, it is reasonable to adopt the laboratory fissure strengths for design purposes. If the fissures have a distinct preferred orientation(s) then laboratory fissure strength may be used only in this orientation(s), with softened or possibly peak strength in other directions.

It is practicable to quantify fissure orientation and continuity if these are mapped in excavations, shafts or pits. The authors have successfully used stereonet methods for plotting orientation of fissures in basaltic and other clays, in a similar way to plotting joints. Where fissures are non continuous, some value between fissure and peak strength should be adopted, depending on the relative proportion of fissured and intact soil on the failure surface being analysed. This is usually very difficult to assess and invariably requires a degree of judgement.

The authors' experience is that where fissures are polished, continuous with preferred orientation, adoption of the fully softened strength as suggested by Chandler (1984) is not sufficiently conservative, and a strength between fully softened and residual strength is necessary to explain failures which have occurred.

On a practical note, it is very unusual to be able to see the fissures in soil samples prior to testing, so it is usually necessary to test a large number of samples to obtain sufficient samples which fail on fissure surfaces not intersecting the end plates. Some fissured samples tend to fall to pieces on ejection from a sample tube. This can be overcome to a degree by cutting down the sides of the sample tube, so the sample can be lifted out, rather than forced out the ends in the normal manner.

6.1.6 *Determination of Skempton's A and B coefficients from triaxial tests*

When designing earthfill dams greater than about 20 or 30 m high, it is necessary to consider pore pressures which build up in the earthfill during construction. These may determine the factor of safety of the upstream slope.

Skempton (1954) related the change in pore water pressure Δu with changes in the principal stresses in the soil by the relationship

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$$

This expression is often rewritten in the form

$$\frac{\Delta u}{\Delta\sigma_1} = \bar{B} = B\left[\frac{\Delta\sigma_3}{\Delta\sigma_1} + A\left(1 - \frac{\Delta\sigma_3}{\Delta\sigma_1}\right)\right]$$

In most applications it is sufficiently accurate to assume that $\Delta\sigma_3 / \Delta\sigma_1$ remains constant, and that $\Delta\sigma_v$ = changes in vertical total stress $= \Delta\sigma_1$. Then from the above equation, $\Delta u = \bar{B}\,\Delta\sigma_v$.

A and B are obtained from laboratory triaxial tests by:

a) monitoring the change in pore pressure Δu resulting from a change in cell pressure. Since $\Delta\sigma_1 = \Delta\sigma_3$, $\Delta\sigma_1$-$\Delta\sigma_3 = 0$ and B can be determined from $\Delta u = B\,\Delta u_3$. Note that as discussed in Section 8.1.5, B is not constant for partially saturated soils, approaching 1.0 as the sample becomes saturated.

b) monitoring the change in pore pressure resulting from a change in deviator stress $\Delta\sigma_1$-$\Delta\sigma_3$, with cell pressure constant ($\Delta\sigma_3 = 0$). This is commonly done under conditions of zero lateral strain, i.e. Ko conditions, to simulate such condition in the field. Head (1985) gives details of test apparatus which allows this. Usually the value of A at 'failure' (A_f) is used in analysis. 'Failure' is usually determined at maximum deviator stresses but may be taken at maximum principal stress ratio.

This approach ignores the effect of the intermediate principal stress, and A for plane strain conditions is not the same as for triaxial conditions.

Henkel (1960) gives a method which accounts for the intermediate principal stress. Wroth (1984) relates A to the critical state parameters and shows that A can be predicted from these parameters. In most practical cases Skempton's approach is adequate.

The analysis of the stability of earthfill embankments for the 'construction' condition is discussed in more detail in Chapter 10.

6.1.7 *Direct and ring shear testing – test procedures and limitations*

6.1.7.1 *Test procedures and application*

Direct shear tests are the most common method of obtaining the residual strengths of granular soils, e.g. sand, silt, gravel and of existing planes of weakness in the soil, e.g. slide planes and fissures. Direct shear tests are also used to obtain peak strengths of cohesive soils but this is more commonly done by triaxial test.

Ring shear equipment is less commonly available but is, in many cases, a more reliable way of determining the residual strength of clay soils. Tests are usually saturated consolidated drained (CD) with the sample sheared at a slow constant rate of displacement, so that pore pressures due to shearing are dissipated giving drained conditions. Details of test procedures are given in Bishop et al. (1971), Bromhead (1979), Saada & Townsend (1981) and Bromhead & Curtis (1983).

6.1.7.2 *Use of direct shear to assess residual strength*

True laboratory residual strength of clay soils only develops with significant displacement. Skempton (1985) gives the 'typical displacement values' reproduced in Table 6.1.

Most laboratory shear box equipment is 75 × 75 mm or 60 × 60 mm, and the maximum practical displacement is of the order of 6 to 10 mm, sufficient only to measure the peak and possibly the softened strength.

'Residual' strength is therefore obtained with a direct shear machine by repeatedly shearing the sample until the strength is not further reduced by further shearing. Typical load displacement curves for 'turbulent' and 'sliding' shear are shown in Figure 6.17.

The repeated shearing is achieved in several ways:

– shear to limit of travel, unload (partly), rewind rapidly to start, then reload and reshear etc until a minimum is reached, or

– shear to peak, then unload partly, and wind back and forth rapidly until a total travel of say 50 mm is achieved, reload and reshear etc until a minimum is achieved, or

Table 6.1. Typical displacements at various stages of shear in clays having clay fraction >30% (Skempton 1985).

<table>
<tr><th rowspan="2">Stage</th><th colspan="2">Displacement mm(2)</th></tr>
<tr><th>Overconsolidated</th><th>Normally Consolidated</th></tr>
<tr><td>Peak</td><td>0.5–3</td><td>3–6</td></tr>
<tr><td>Rate of volume change approx zero(1)</td><td colspan="2">4–10</td></tr>
<tr><td>At $\phi'_R + 1°$</td><td colspan="2">30–200</td></tr>
<tr><td>Residual ϕ'_R</td><td colspan="2">100–500</td></tr>
</table>

Notes: (1) i.e. fully softened strength.
(2) For ϕ'_n <600 kPa.

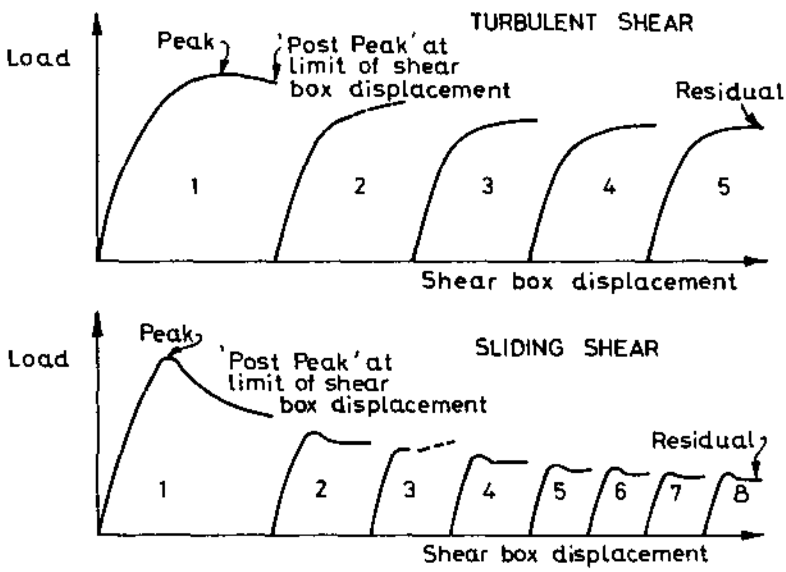


Figure 6.17. Typical load displacement curves for direct shear tests (Skempton 1985).

– shear back and forth under load at a slow strain rate until a minimum is achieved.

Further acceleration of achievement of the residual strength is sometimes accomplished by cutting a slide plane into the sample prior to testing. The cut surfaces are sometimes polished on a glass plate prior to testing.

Problems which arise in the use of the direct shear to assess residual strength include:

– Repeated reversal of the direction of shearing is necessary, and this may partially destroy the alignment of particles on the shear plane, preventing a true residual strength being achieved.

– It is difficult to ensure that the sample is properly saturated. Unlike triaxial testing, pore pressure cannot be measured, so reliance has to be made on monitoring the consolidation deformation of the sample during saturation and consolidation. The best approach is to saturate under a low confining pressure until swell (or consolidation) ceases, then load to the first test confining pressure, and monitor until settlement ceases. This will usually produce slightly

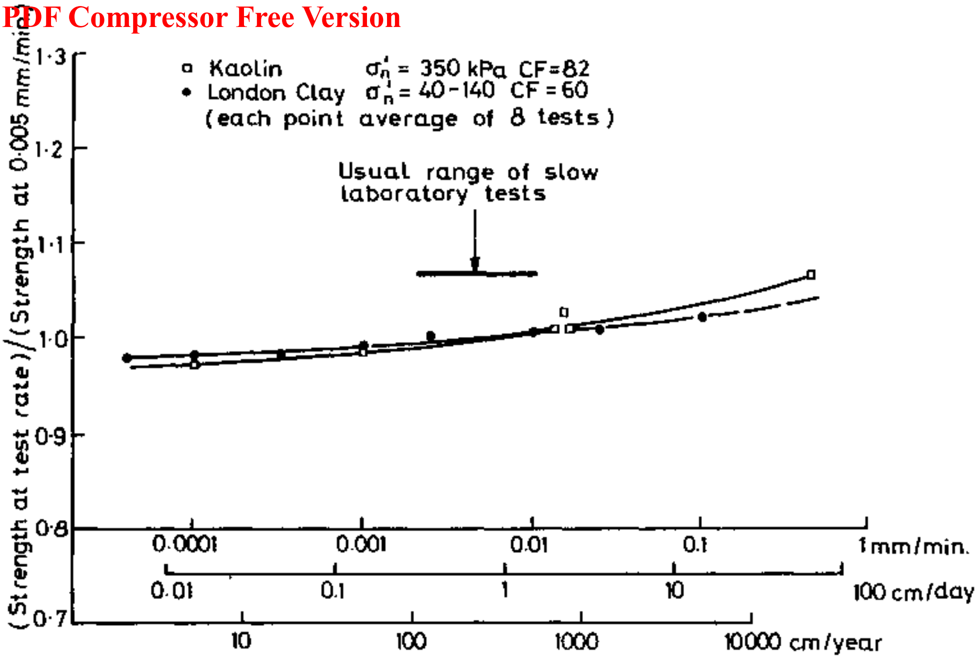


Figure 6.18. Variation in residual strength of clays at slow rates of displacement (Skempton 1985).

conservative results as the soil has been allowed to swell more than it would in the field.

– Testing at a too high strain rate leads to overestimation of shear strength in a similar way to triaxial testing, i.e. c' is overestimated, ϕ' underestimated. This can be overcome by monitoring the consolidation to obtain t_{50}, t_{90} or c_v value, and then using formulae developed by Akroyd (1975), Bowles (1978) or Head (1985) to estimate the strain rate. Skempton (1985) showed that the answer obtained is not greatly sensitive to strain rate provided reasonably low strain rates are used. This fact may be used to speed up test rates, provided that when a constant shear stress has been reached the strain rate is reduced, and the resulting shear stress is not significantly lower than the previous minimum (see Fig. 6.18).

– Testing at too high a confining stress range leads to an overestimation of c', underestimation of ϕ' due to curvature of the strength envelope in the same way as for triaxial testing.

– Repeated reversal of the shear box often leads to soil squeezing out between the two halves of the box. If this happens, or if the box tilts, interference can occur between the two halves of the box leading to erroneous results.

Apart from the practical problems outlined above there are theoretical limitations with the shear box. These are summarized by Saada & Townsend (1981).

6.1.7.3 *Use of ring shear to assess the residual strength of clay soils*

To overcome the problems of multiple reversals of a shear box to obtain sufficient displacement on the slide plane to achieve residual strength, Bishop et al. (1971) developed a ring shear apparatus.

In the ring shear, an annular ring shaped specimen (Fig. 6.19) is subject to a constant normal stress σ'_n confined laterally, and caused to shear on a plane of relative rotary motion at a constant rate of rotation.

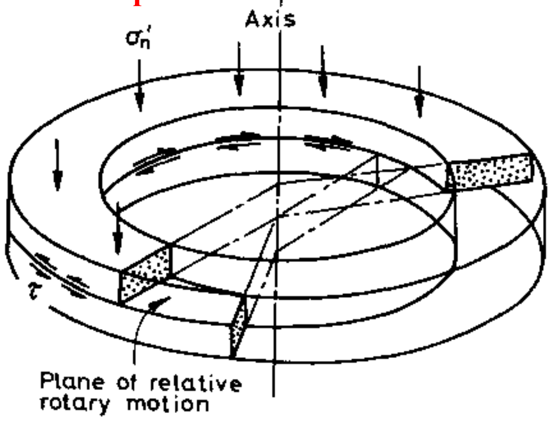


Figure 6.19. Ring shear test sample.

For the Bishop et al. (1971) machine (now manufactured by Wykham Farrance) the sample is 150 mm outside diameter, 100 mm inside diameter, and 19 mm thickness. Remoulded samples are usually tested, but undisturbed samples can be placed in the equipment. The machine is mechanically complex and expensive, putting it largely in the research category. Bromhead (1979) and Bromhead & Curtis (1983) describe a simpler ring shear device. Bromhead (1986) gives some practical details on how to operate the equipment. The sample in the Bromhead machine is outside diameter 100 mm, inside diameter 70 mm and only 5 mm thick.

The ring shear devices have essentially unlimited 'strain' availability, so overcoming the major objection to the reversing direct shear test. However, there are limitations:

– only remoulded samples can be tested,

– only the residual strength can be obtained. The peak strength is affected by remoulding and non uniform stress conditions,

– the sample may tend to squeeze out the sides of the ring. The Bishop machine is constructed to overcome this and can be used for softer clays than the Bromhead machine,

– saturation and testing at a strain rate appropriate to give the required drained conditions is necessary as for the direct shear test.

6.1.8 *Comparison of field residual with laboratory residual strength obtained from direct shear and ring shear*

There have been many papers published comparing field residual strengths obtained by back-analysis of landslides, and direct shear testing of the undisturbed slide plane, with residual strengths obtained from multiple reversal direct shear, and ring shear tests:

Bishop et al. (1971) showed that multiple reversal shear box tests gave significantly higher 'residual' strengths than did the ring shear. These figures show reversing direct shear to be 3 to 4° above the ring shear. Direct shear and ring shear gave similar values for shale (very weak rock). Cut planes were used on the shale.

Townsend & Gilbert (1973) showed that ring shear and repeated reversal direct shear give similar results for a range of soft rocks.

Chandler (1984) shows that the ring shear device underestimates field strengths, and suggests direct shear tests on undisturbed slide planes themselves as being the best method to obtain the field residual strength.

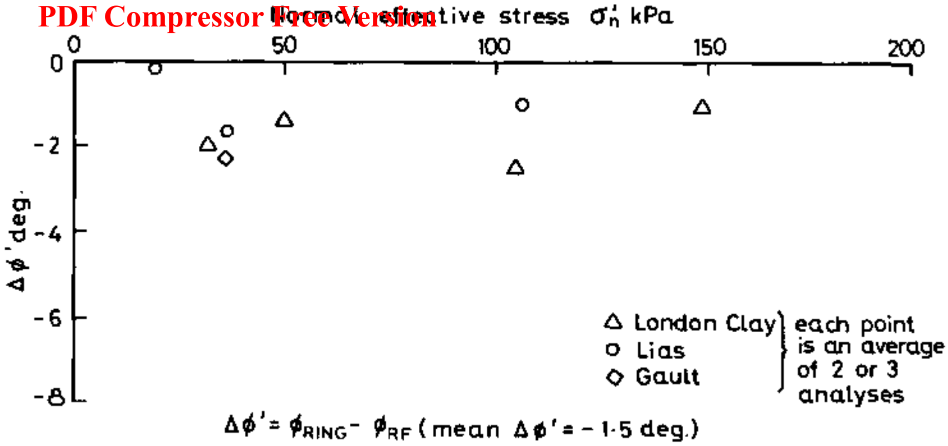


Figure 6.20. Difference between ring shear and field residual strength (Skempton 1985).

Bromhead (1979) and Bromhead & Curtis (1983), present results using the Bromhead ring shear, the Imperial College (Bishop et al.) ring shear and shear box tests on slide surfaces. These show that a) there is good agreement between the Imperial College and Bromhead machines; b) the ring shear results on clay in which failure occurred, correlate well with the direct shear tests on the slide plane (ring shear slightly conservative at lower stress levels).

Skempton (1985) indicates that the ring shear device underestimates residual strength compared to field residual (ϕ'_{RF}) by 1 or 2°. These results are reproduced in Figure 6.20.

In summary the literature indicates:

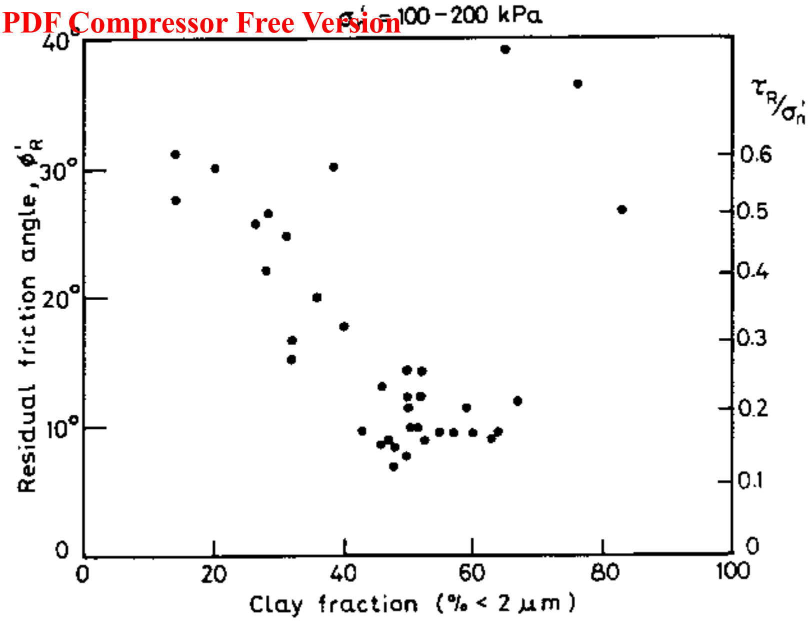
– direct shear tests on slide plane or bedding plane shear are the most reliable indicator of field residual strength;

– ring shear devices either underestimate by 1 to 2° or approximate the field residual,

– multiple reversal direct shear on clays will probably overestimate field residual by 1 or 2°. The literature indicates that the difference between multiple reversal direct shear and field residual is only small in weak rocks such as shales and claystones; but the authors' experience in testing clays and very weak claystone is that the difference may be significantly greater (up to 6°), and direct shear tests on the slide plane or bedding plane shear should always be done if practicable.

The authors' experience in testing soil from several landslides in soils derived from sedimentary rocks, tuff and basalt is that the Bromhead ring shear is very susceptible to the presence of particles of sand. These 'catch' in the rings and lead to higher residual strengths than obtained from multiple reversal direct shear, or tests on slide planes. Comparable results were only obtained by drying and sieving the soil to remove any particles of sand before testing.

It can be seen that ideally tests should be carried out on the low strength planes if already present, or that ring shear tests should be carried out provided that the soil is free of sand particles. Unfortunately, ring shear machines are not widely available and reversing direct shear tests have to be used. In these cases, the design strength should be selected at the lower bound of results unless exhaustive testing shows this to be conservative. This is valid since few (if any) of the reversing tests may have achieved a proper alignment of particles, and the low results (provided these were not testing errors), are probably the best indicator of residual strength. The



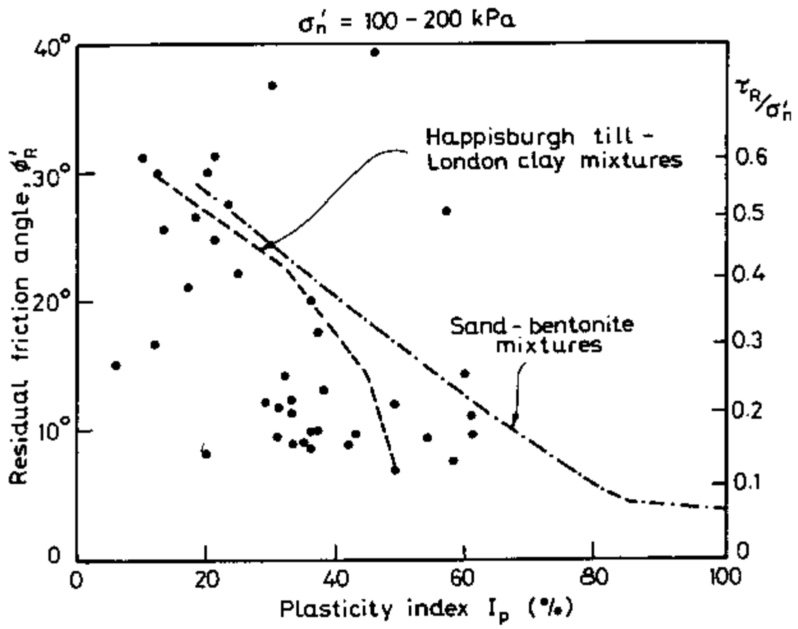


Figure 6.21. Relationships between residual friction angle, clay fraction and plasticity index (Lupini et al. 1981)

validity of the results should also be checked by comparison with published data (see Section 6.1.9).

In many cases of landsliding, where stabilization measures are being designed, the strength can be determined by back analysis, and stabilization works designed on the basis of increasing the factor of safety by no less than 30%. This relieves some of the need for accurate knowledge of the shear strength from laboratory tests.

A method which has been used by the authors to determine the 'lower bound' residual strength of a soil, is to separate the clay fraction from the silt (and sand) of the soil by sedimentation. The clay fraction is then tested by overconsolidating it in the direct shear, and then determining the residual strength by multiple reversal direct shear testing. This procedure was suggested by Pells (1982) and has given reasonable values when used.

In nature, it is not uncommon to observe a clay rich zone 0.2 mm to 1mm thick at the slide surface, and in these circumstances the procedure outlined above can be expected to give a good representation of the field condition.

6.1.9 *Correlations between residual friction angle and other soil properties*

Given the difficulties in assessing the residual strength of soils, it is useful to check the order of magnitude of the laboratory test results by use of published information relating residual friction angle and other soil properties.

Lupini et al. (1981) presented plots of ϕ'_R clay fraction and plasticity index. These are reproduced in Figure 6.21.

The notable exceptions to the plots are allophane and halloysite clays. Skempton produced a similar graph relating to clay fraction, reproduced in Figure 6.5.

Mesri & Cepeda-Diaz (1986) presented graphs using data from Kenney (1967) and tests they carried out on clay shales. These are reproduced in Figure 6.22.

They note that the method of sample preparation (degree of grinding, use of dispersants) affects the clay fraction measurement.

6.2 SHEAR STRENGTH OF ROCK

The shear strength of rock can be considered in two parts:

– the strength of the rock substance,

– the strength of the rock mass, as determined by the strength of joints, bedding planes and other discontinuities.

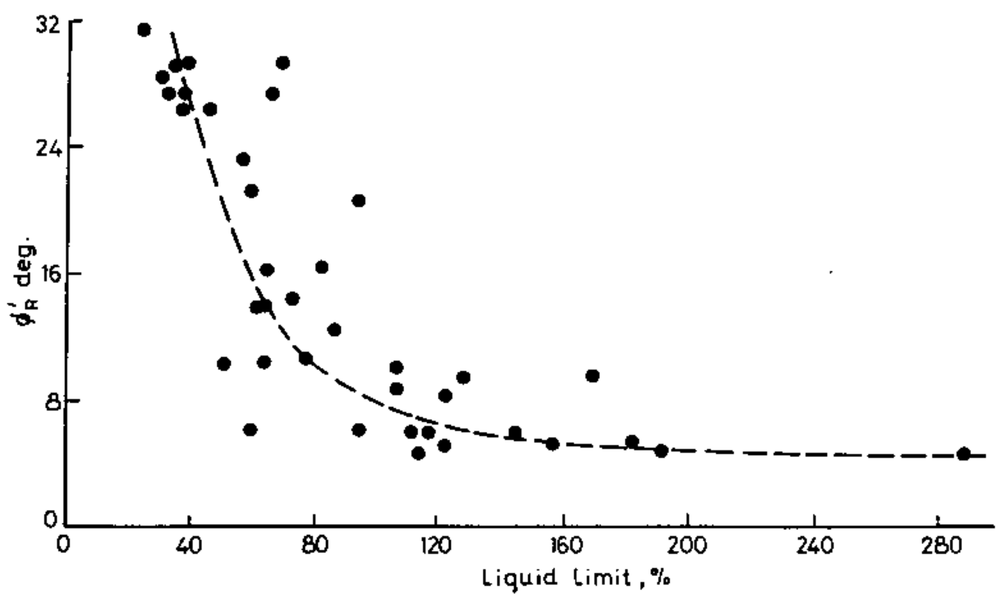
For most dam embankments, the foundation is excavated to a level so that the foundation strength (i.e. the mass strength) is not a controlling factor in the stability of the embankment, this being determined by the strength of the embankment materials. However there are exceptions to this rule, particularly where bedding plane shears are present, usually in interbedded strong and weak sedimentary rocks, e.g. at Sugarloaf Dam (Casinader & Stapledon 1979).

The rock substance strength is not a factor in stability of embankment dams, as it is invariably high compared to the strength of embankment materials. However rock substance strength is an important factor when assessing excavatibility of rock in the embankment foundation, spillway, diversion tunnels etc.

The shear strength of rock joints, bedding planes etc. is important when considering the stability of cuttings in rock for the embankment, spillway, intake, diversion tunnel etc. How-even in other than major cuttings, it would be normal to estimate the strength parameters, rather than carry out any laboratory testing.

The following discussion is therefore limited to consideration of methods of measuring and classifying the rock substance strength, and a brief overview on measurement of the strength of discontinuities in rock.

For a more detailed discussion on shear strength of rock, readers should refer to Hoek & Bray (1981), McMahon (1985) and Hoek & Brown (1980a, b).



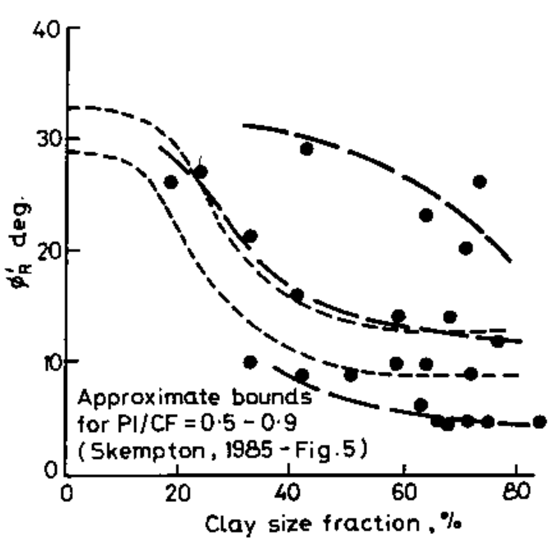


Figure 6.22. Relationships between residual friction angle, liquid limit, clay fraction and peak friction angle (Mesri & Cepeda-Diaz 1986).

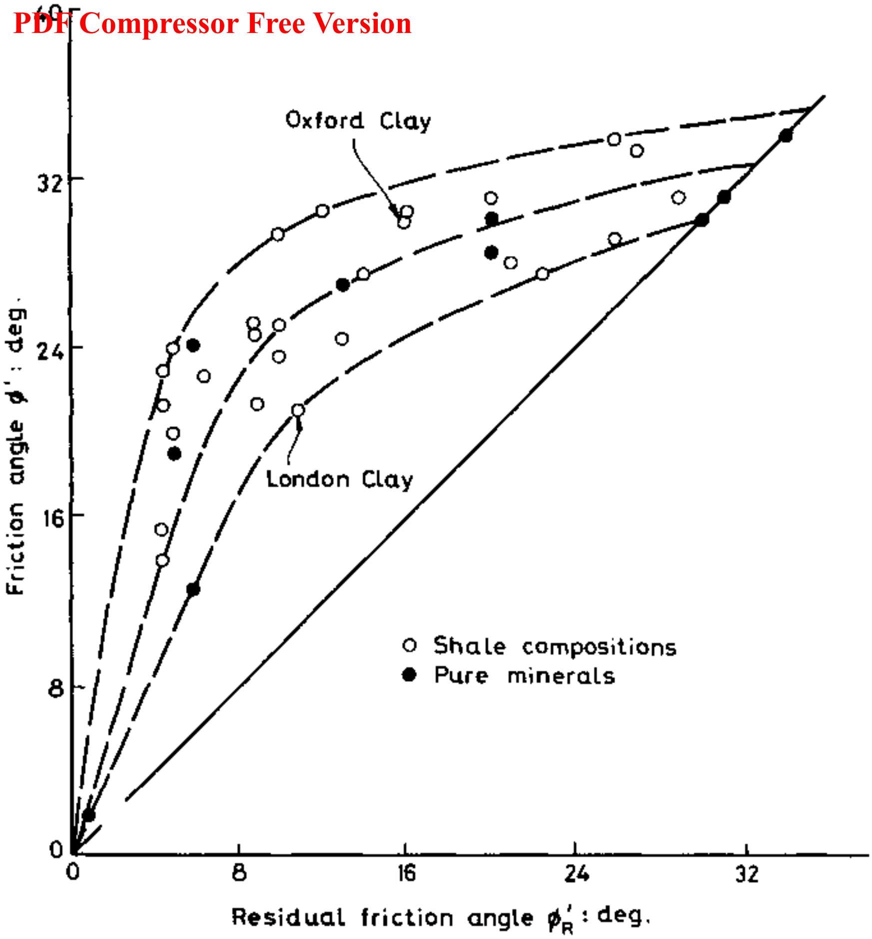


Figure 6.22 (continued).

6.2.1 *Rock shear strength parameters – Definitions*

6.2.1.2 *Shearing on a defined discontinuity*

When considering shearing on a single joint, bedding plane or other discontinuity in rock, or a series of parallel features, such that failure does not involve shearing of intact rock, a Mohr Coulomb type of failure criterion is appropriate, i.e.

$$\tau = c' + \sigma'_n \,\mathrm{Tan}\,(\phi' + i)$$

where τ = shear stress on the surface

σ'_n = effective normal stress (after allowing for pore pressure)

c' = effective cohesion

ϕ' = effective friction angle of the smooth surface of the discontinuity

i = inclination of large scale roughness on the discontinuity.

It should be noted that the effective cohesion c′ arises because of curvature of the strength envelope, and even more than for testing of soil (Section 6.1.4) it is important that the tests be carried out over the appropriate range of normal stresses to be experienced in the field, and the strength envelope (as defined by c′,ϕ′) be selected within the correct range.

As discussed in McMahon (1985) the question arises as to whether ϕ′ should be the peak friction angle, residual friction angle or something in between and how should this be measured. How i is measured, and whether large and small scale roughness should be included is also to be determined.

On the basis of back analysis of several failures in rock slopes, McMahon (1985) concludes that:

– ϕ′ should be the mean laboratory ultimate strength,
– i should be the mean of the relatively large scale roughness angles.

The 'ultimate' strength is the minimum strength obtained by multiple reversal direct shear testing, using representative samples of the discontinuity. It is controlled by grinding of asperities on the surface forming a powder like detritus. McMahon (1985) differentiates this 'ultimate' strength from the 'residual' strength which he assigns to lower strengths which can be produced by special polishing preparation on the laboratory samples, or to lower strengths which may apply to other discontinuities in the same rock type (e.g. clay coated discontinuities). McMahon points out his term 'ultimate' corresponds to the ISRM (1981) term 'residual'.

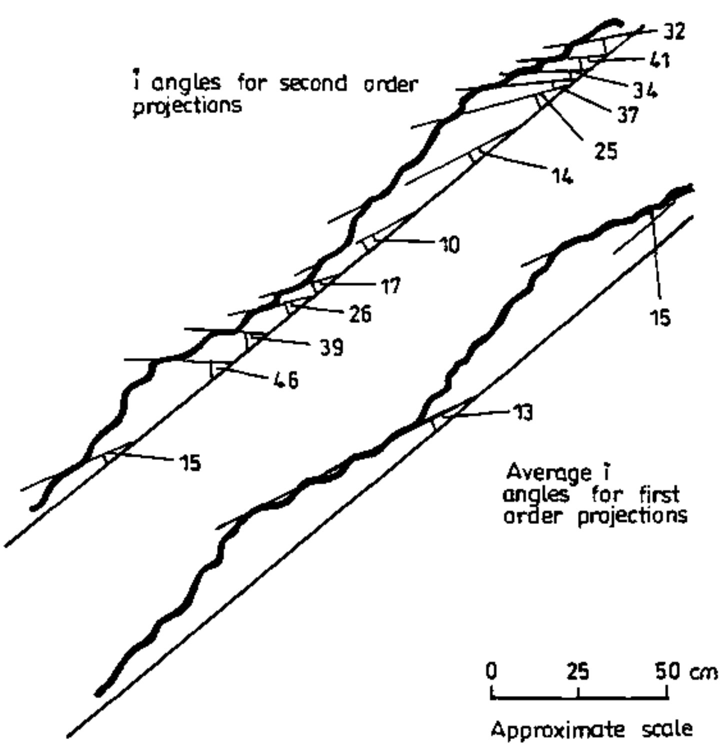


Figure 6.23. Measurement of i angles for first and second order roughness on a rock surface (adopted from Patton 1966).

McMahon (1985) defines 'relatively large roughness' as those with wave lengths equal to or greater than 2% of the potential failure surface. This corresponds to Patton's (1966) 'first order' roughness shown in Figure 6.23.

It should be noted that in this approach second order roughness is ignored, as it is considered not to affect the analysis.

The method described above is a practical one found by McMahon (1985) to give reasonable results. Mostyn (1988) has worked on several of the projects described by McMahon and has extensively reviewed the topic of rock shear strength, prefers the McMahon approach with the qualification that the method requires judgement and experience to apply successfully. Readers will find other opinions from experienced practitioners and provided the methods have a sound practical basis they may give satisfactory results.

6.2.1.2 *Shearing of a closely jointed rock mass*

When considering shearing a rock mass with a number of joint sets, and when the joint spacing is close in relation to the size of slope being analysed, the behaviour may differ significantly from that outlined above.

For these circumstances, it is recommended that the method of Hoek & Brown (1980a, b) described in Hoek & Bray 1981) be adopted. This is an empirical formula:

$$\tau = A\sigma_c\,(\sigma/\sigma_c - T)^B$$

where τ = shear strength

σ = normal stress

σ_c = uniaxial compressive strength of the intact rock pieces.

A and B are constants defining the shape of the Mohr failure envelope

$$T = \tfrac{1}{2}\left(m - \sqrt{m^2 + 4s}\right)$$

where T is the normalized uniaxial tensile strengths and m and s are dimensionless constants which depend on the shape and degree of interlocking of the individual pieces of rock within the mass.

Hoek & Bray (1981) give details on how to determine these constants from laboratory tests. Hoek & Brown (1980a) give details on how to determine the constants from rock mass classification.

6.2.2 *Laboratory testing of rock discontinuities*

The shear strength of rock discontinuities can be determined in the laboratory by use of direct shear equipment similar to that used for testing of soils (or larger purpose built direct shear machines), or by use of a portable shear machine, e.g. Robertson Research machine or similar shown in Figure 6.24.

There are many variations of this portable equipment (often called a Hoek shear box).

The following points are made in respect to shear testing of rock discontinuities:

– Tests should be carried out with normal stresses similar to those to be experienced in the field. This may be difficult with the Robertson Research Machine because, with the portable equipment, it is difficult to test at low stress levels.

– Tests should be carried out on smooth surfaces of the discontinuity to limit the influence of

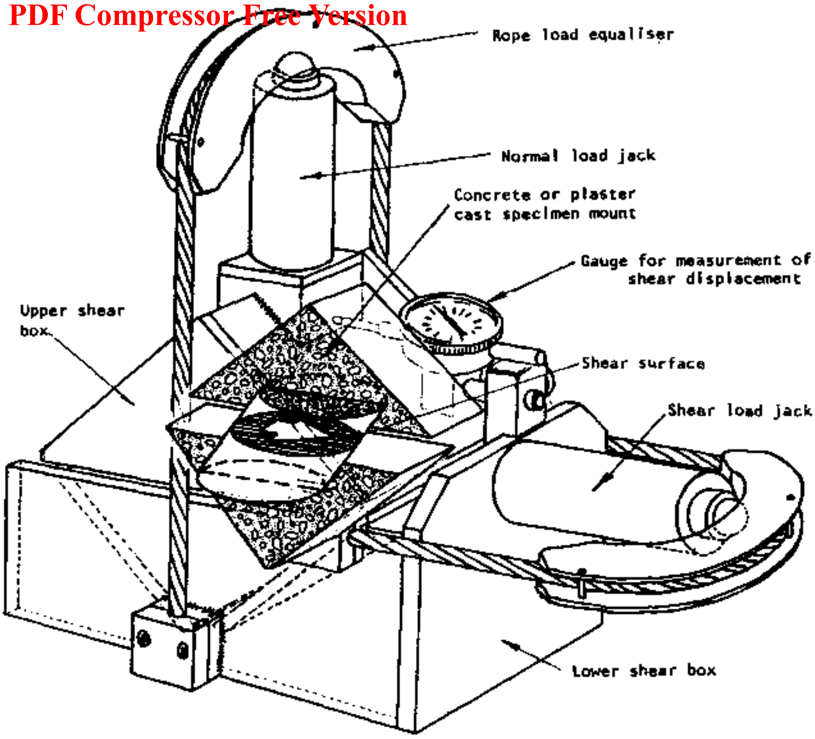


Figure 6.24. Portable direct shear machine (Hoek & Bray 1981).

secondary roughness effects, otherwise these would lead to a large amount of scatter of the results making separation of ϕ' and i effects difficult.

– Conventional direct shear machines for testing soil have a limited range of normal stresses which can be applied. Testing at too low a stress range will overestimate the shear stress, because of the marked curvature of the Mohr envelope in most cases.

– Tilting of the sample in the equipment is a problem in both cases and results in an overestimation of strength.

– With the Robertson Research Machine, it is difficult to maintain a constant normal stress because the cables which apply the normal stress prevent dilation during shearing. Hencher & Richards (1989) describe a machine which overcomes this problem, and allows testing of drill core with normal stresses up to 2 MPa.

When testing weak rocks, in particular those susceptible to softening, the samples should be allowed to 'saturate' by soaking under a small normal stress (say 20 kPa). In a direct shear machine, any swell which may result can be observed.

ISRM (1981) detail test methods to be used with shear box equipment.

6.2.3 *Laboratory and field testing of rock substance strength*

The measurement of the strength of rock substance is based on the unconfined compressive strength test.

UCS testing of a rock is usually carried out on drill core with a height to diameter ratio of 2,

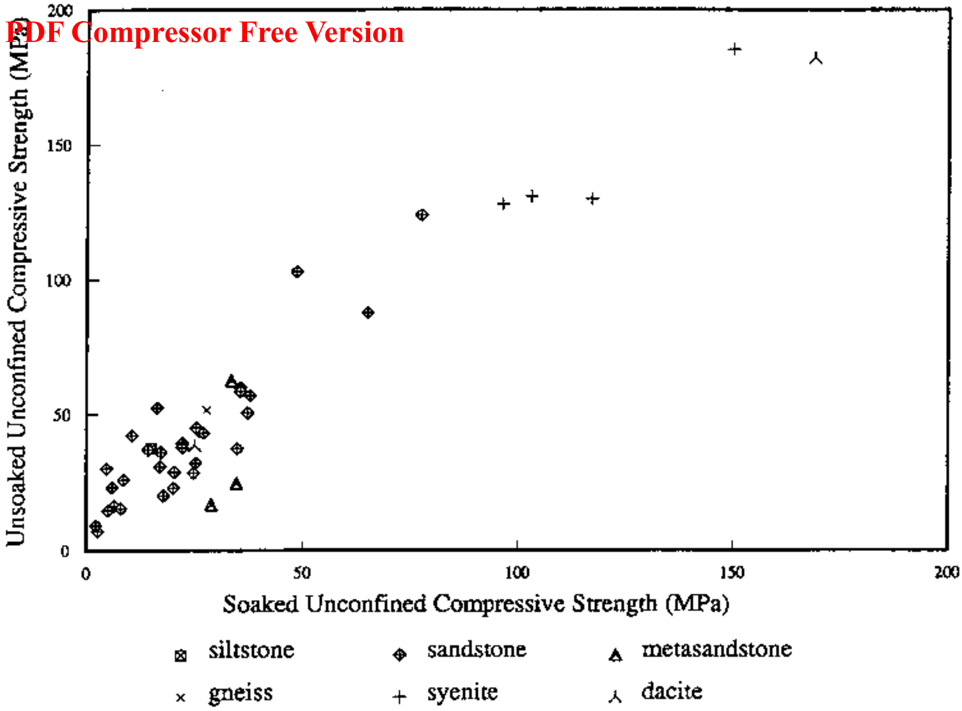


Figure 6.25. Relationship between soaked and unsoaked unconfined compressive strength.

but may have to utilise material in the shape which is recovered during the site investigation. Sample preparation is affected by inhomogeneity and the presence of banding or foliation. The strength of some rocks is significantly affected by moisture content and restoration of field moisture conditions in the laboratory, after drying out, is difficult to achieve.

Figure 6.25 shows a comparison between soaked and unsoaked unconfined compressive strengths for several rocks. It can be seen that the soaked strengths are lower, by varying amounts. Some rocks, e.g. some shales and claystones will slake on rewetting, leaving no core to test in the unconfined test. In these cases it is essential that the field water content is preserved.

It has been recognised that in many situations insufficient UCS strength determinations could be obtained to effectively assess site conditions, and the Point Load strength test was developed to:

– provide a test which enabled field classification of rock strength,
– allow testing at field moisture content,
– provide a classification using samples of different shape without extensive preparation,
– allow testing at close spacing to confirm substance strength uniformity and differences.

The Point Load strength test is based on the indirect tensile strength test of rock core by fracturing a cylinder of rock, using a machine equipped with conical platens and a load measuring system.

The method was originally described by Broch & Franklin (1972), then adopted by the International Society for Rock Mechanics (1972, 1985).

The test provides a Point Load Strength (I_s) for different size samples which are then corrected to a standard 50 mm size equivalent, the Point Load Strength Index $I_{s(50)}$ for comparison with other test results.

There are two main types of test:

– The diametral test in which a cylinder of rock, usually core, is loaded normal to its axis.

– The axial block or irregular lump test where either loading is parallel to the core axis (axial), or the sample is not cylindrical in shape.

The difference between the tests is that results to the axial, block and irregular lump tests have to be corrected for shape to an equivalent diameter cylinder in order to give comparable strength values.

If possible, samples should be tested at natural moisture content as soon as practicable after drilling. Dried samples can give much greater strengths than most samples.

The Point Load Strength Index $I_{s(50)}$ is grouped into the following strength classes:

$I_{s(50)}$ MPa	Class	Abbreviation
>10	Extremely high	EH
3 to 10	Very high	VH
1 to 3	High	H
0.3 to 1	Medium	M
0.1 to 0.3	Low	L
0.03 to 0.1	Very low	VL
<0.03	Extremely low	EL

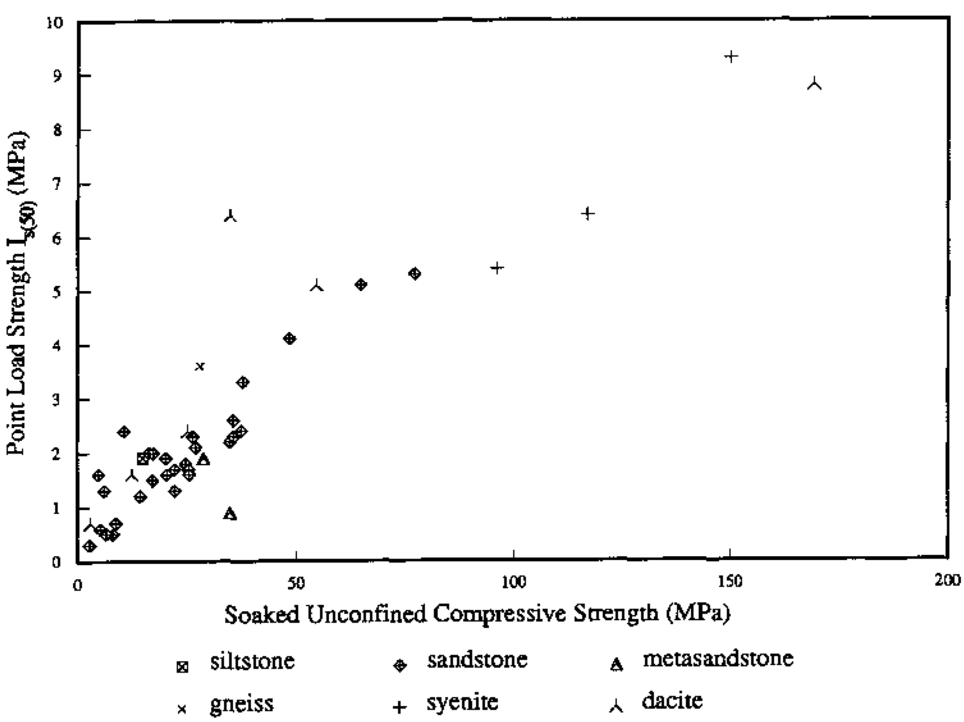


Figure 6.26. Relationship between unconfined compressive strength and Point Load Strength Index.

Where the rock substance is banded the degree to which the banding affects the substance strength can be presented as the Anisotropy Index $I_{a(50)}$ where:

$$I_{a(50)} = \frac{I_{s(50)} \text{ normal to banding}}{I_{s(50)} \text{ parallel to banding}}$$

There is no fixed relationship between unconfined compressive strength and Point Load Strength Index. A ratio of $UCS/I_{s(50)} = 24$ is commonly adopted for 'N' size core (54 mm diameter). The constants for other core sizes are: 60 mm-24.5; 50 mm-23; 30 mm-19; 20 mm-17.5 (Hoek & Bray 1981). However, considerable scatter occurs in practice as shown in Figure 6.26.

In view of this, it is recommended that the relationship be established for the different rocks on a site by doing some (soaked) unconfined compressive strength and Point Load Strength Index tests.

6.3 PERMEABILITY OF SOILS

The 'permeability,' or more correctly 'permeability coefficient' or 'hydraulic conductivity' of the soil in a dam embankment or foundation, is often measured in the laboratory on either 'undisturbed' or remoulded samples of the soil.

The permeability is not a fundamental property of the soil but depends on a number of factors. Head (1985) outlines these as:

- particle size distribution,
- particle shape and texture,
- mineralogical composition,
- void ratio,
- degree of saturation,
- soil fabric,
- nature of fluid,
- type of flow,
- temperature.

In embankment dam engineering these factors have varying degrees of influence:

Particle size distribution. The permeability is dependent on the particle size distribution of the soil with fine grained clay soils having permeabilities several orders of magnitude lower than that of coarser soils, i.e. sands and gravels. Figure 6.27 shows in general terms the range of permeabilities which can be encountered.

It has been recognised that the finer particles in a soil largely determine its permeability, and soils are often compared by their 'effective grain size', D_{10}, which is the particle size for which 10% of the soil is finer.

Particle shape and texture. This affects permeability to a lesser extent. Elongated particles have a lower permeability than rounded, and rougher textured particles have a lower permeability than smooth.

Mineralogical composition. This is a factor in clay soils. Montmorillonite clays, for example, are finer grained and have a greater tendency to adsorb water (and hence a lower permeability) than say a kaolin clay. The mineralogy of sand and gravel has little effect.

Void ratio. The void ratio of a soil has an important effect on permeability. Cohesive soils which are compacted to a high density ratio (e.g. 98% of standard maximum dry density) will

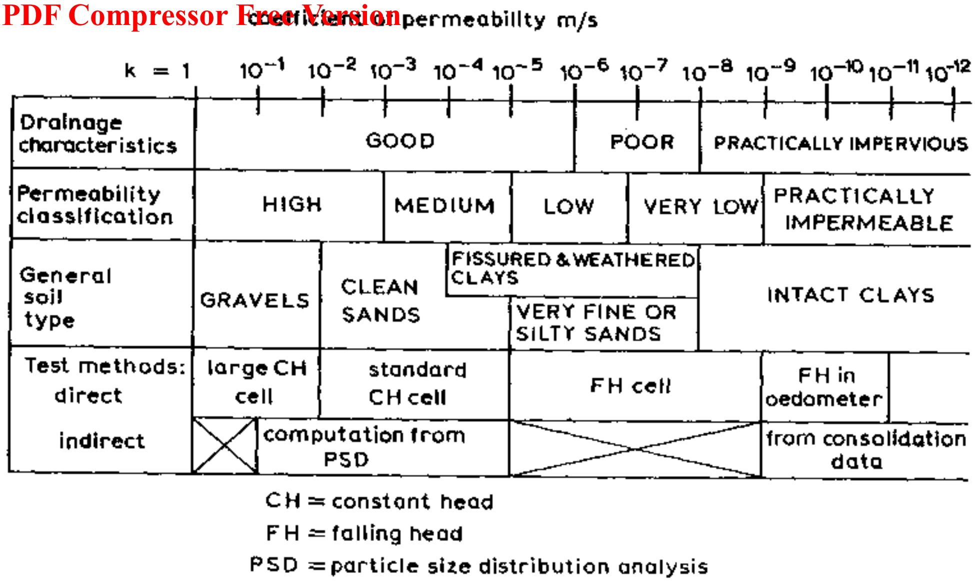


Figure 6.27. Permeability and testing method for the main soil types (Head 1985).

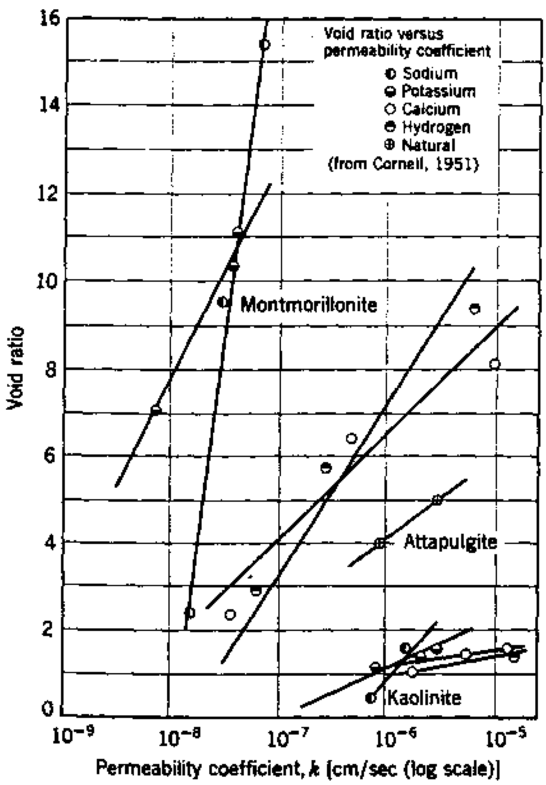


Figure 6.28. Effect of void ratio and clay mineralogy on permeability (Lambe & Whitman 1979). Reprinted by permission of John Wiley & Sons Inc.

have lower permeability than those compacted to a low density ratio (e.g. 90%). The difference may be orders of magnitude. Figure 6.28 shows the effect of void ratio for several clays.

The void ratio also has an effect on the permeability of granular soils, with soils compacted to a small void ratio (dense) having lower permeability than those with a high void ratio (loose). This effect is allowed for in the Kozeny-Carman formula discussed below.

Degree of saturation. When a soil becomes partially saturated, the permeability is reduced. This occurs due to a reduction in the total cross section of pores filled with water. These water filled pores tend to be the finer pores, because water is most readily removed from the larger pores which have low suction potential. The remaining finer water filled pores have naturally lower permeability. The partially saturated permeability may be an order of magnitude lower than the saturated permeability.

Soil fabric. The fabric of a soil can have a major effect on the permeability. This is particularly important for soil masses *in situ*, where stratification or layering of different soil types, e.g. sand, silt, clay; can lead to markedly different permeability along the strata to that across the strata. Ratios of horizontal to vertical permeability (kH/kV) of 100 or more are not uncommon.

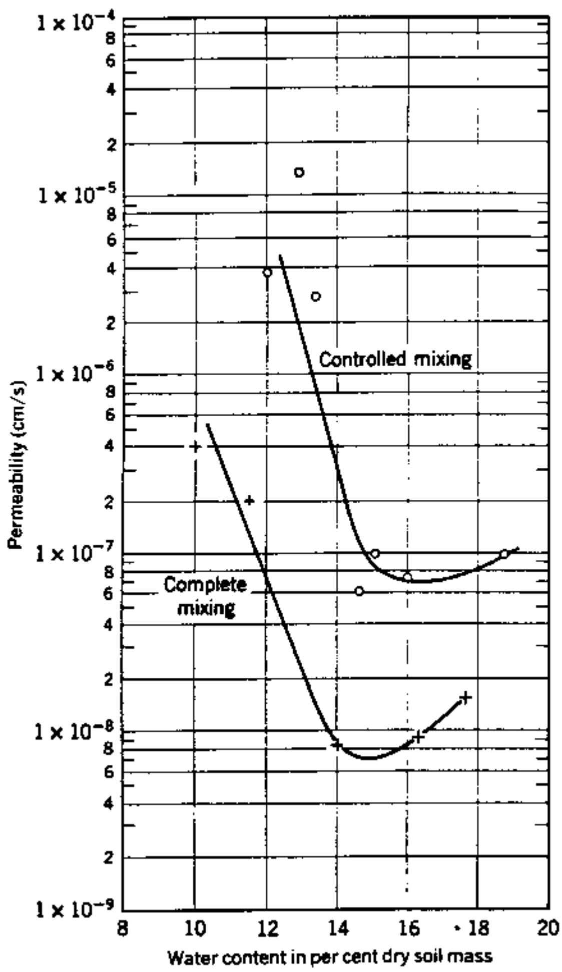


Figure 6.29. Effect of structure on permeability (Lambe & Whitman 1979). Reprinted by permission of John Wiley & Sons Inc.

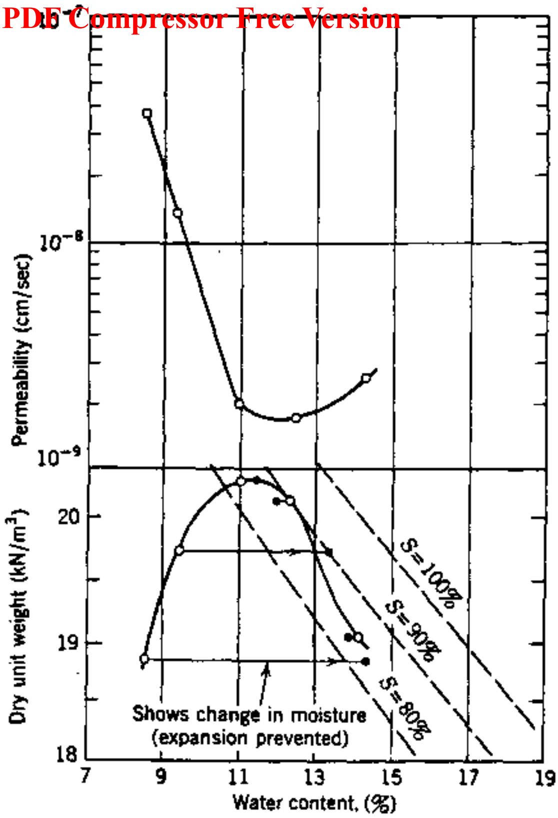


Figure 6.30. Effect of compaction water content on permeability (Lambe & Whitman 1979). Reprinted by permission of John Wiley & Sons Inc.

Relic jointing, fissures, root holes, worm holes, lateritisation etc all influence the soil fabric and lead to much higher permeability for the soil mass than the soil substance permeability.

When testing samples from such soil masses in the laboratory, it is essential that the presence of fabric is allowed for, e.g. in orienting samples along or across the strata, or in taking sufficiently large samples to include representative fissures. Rowe (1972) suggests that samples of up to 250 mm diameter may be needed to correctly sample fabric. Generally speaking if *in situ* testing can be carried out, it will be more reliable than laboratory testing.

The permeability of recompacted soil is also affected by the soil fabric, which is dependent on the method of sample preparation. Lambe & Whitman (1979) describe this as micro-structure or micro-fabric effects. Figure 6.29 shows the effect of mixing of the soil before compaction, and of adding a dispersant (0.1% polyphosphate) to the soil. In the latter the dispersant breaks up the flocculated structure, resulting in a lower permeability.

The water content at which a clay is compacted affects its permeability. In particular cohesive soil compacted dry of optimum results in a less oriented structure, with potentially aggregates of soil separated by voids, leading to relatively high permeability. An example of this behaviour is given in Figure 6.30.where soils are compacted with the same compactive effect at different water contents.

Nature of fluid. The permeability (coefficient of permeability k) is dependent on the properties of the permeating fluid. If the permeability is related to the absolute (also known as intrinsic, or specific) permeability, K, by

$$k = \frac{K\rho g}{\mu}$$

where ρ = fluid density
g = acceleration due to gravity
μ = fluid viscosity.

For most dam projects water is the permeating fluid and the small variations in viscosity and density, resulting from differences in temperature and dissolved salts content, are not significant compared to other variables, however the chemical compaction of the permeating water can affect soil dispersivity (see Chapter 9) and to a certain extent the permeability.

Type of flow. The basic assumption in calculations using Darcy's Law $q = kiA$ is that the soil is saturated, and the flow is laminar. This is generally the case for flow through and beneath dams but flow in medium and coarse gravels may become turbulent in some cases, eg. around dewatering wells.

6.3.1 *Test methods*

Most permeability testing of soils is carried out using constant head or falling head permeability apparatus. As shown in Figure 6.27 the constant head test is best suited to high to medium

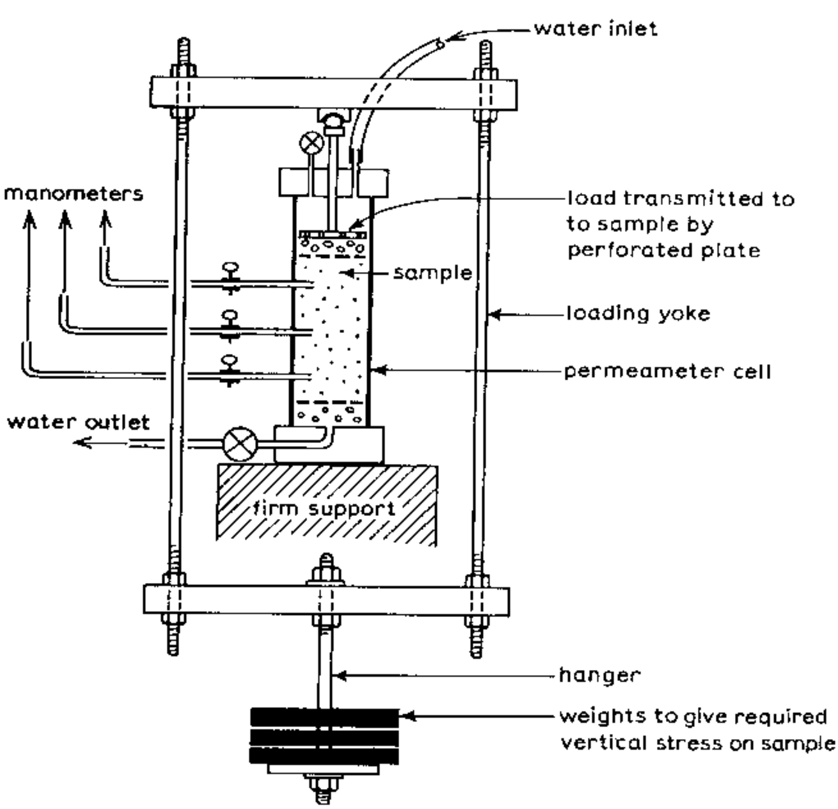


Figure 6.31. Suitable assembly for constant head permeability apparatus (Head 1985).

permeability soils i.e. sand and gravel, and the falling head test to low to very low permeability soils i.e. silts, fine sands and some clays.

For very low permeability soils, the flow rates are too low to practically carry out testing in a conventional permeameter, and it would be more usual to do the testing in an oedometer cell (e.g. a Rowe cell), or to determine the permeability indirectly from consolidation test data.

Many texts in soil mechanics include details of falling head and constant head tests so they are not repeated here. Head (1985) gives detailed instructions on laboratory procedures, as do Standards Association of Australia (1980) and American Society for Testing and Materials D2434. The USBR (1980) also gives detailed instructions for the test in their *Earth Manual*.

Some common errors and problems which can arise with permeability testing include:

a) Leakage past the sample in the test mould.

b) Variability in test results for compacted soils due to difficulty in maintaining uniform water content and density in sample preparation.

c) Failure to saturate sample.

d) Failure to adequately test the fabric of the soil.

e) Testing under little or no confining stress. For dams work in particular it is important to use apparatus such as that shown in Figure 6.31 which allows simulation of the overburden pressure on the soil and in so doing allows testing at the correct void ratio.

f) Excessive flow rates in the apparatus (when testing in coarse sand or gravel) leading to unaccounted losses in the pipework or porous plates at the ends of the sample and an underestimation of permeability.

From the above, a) and c) can be eliminated by careful test procedures; b) is to be expected and requires several tests so that variability can be identified; and d) can be overcome by correct sampling, using larger samples and/or using *in situ* tests. Problems with high flow rates f) can be readily checked by testing the apparatus without soil.

6.3.2 *Indirect test methods*

6.3.2.1 *Oedometer consolidation test*

For 'very low' to 'semi impermeable' clays, it is often more accurate to estimate the permeability from the results of oedometer consolidation tests, rather than to carry out falling head tests. From the theory of consolidation:

$$k = \gamma_w \, c_v \, m_v$$

where γ_w = unit weight of water
c_v = coefficient of consolidation
m_v = coefficient of volume change.

In any oedometer tests c_v and m_v commonly depend on the pressure applied to the soil, so the permeability k also varies depending on the applied pressure. If a Rowe consolidation cell is used with radial drainage (rather than vertical), the horizontal coefficient of consolidation and hence horizontal permeability will be calculated. Details are given in Head (1985).

Similarly, the consolidation phase of a triaxial test can be used to estimate m_v and c_v and hence the permeability. If radial drainage is promoted by surrounding the sample with filter paper, the horizontal coefficients of consolidation, volume change and permeability will be determined.

An advantage of using triaxial testing is that saturation of the sample can be assured by back pressure saturation.

6.3.2.2 *Estimation of permeability of sand from particle size distribution*

An approximate estimate of the permeability of sands and sandy gravels can be obtained from the particle size distribution of the soil. The most commonly used of the available formula are:

(i) Hazen's formula. The most commonly quoted form of the equation is

$$k = C(D_{10})^2$$

where C = factor, usually taken as = 0.01

D_{10} = effective grain size in mm

k = permeability in m/sec.

It should be noted that Hazen's formula was developed for clean sands (less than 5% passing 75 μm) with D_{10} sizes between 0.1 and 3.0 mm. Even within these constraints C is quoted to vary from 0.004 to 0.015 (Holtz & Kovacs 1981, Head 1985). Lambe & Whitman (1979) show values of C varying from 0.0001 and 0.004 with an average of 0.0016 for a range of soils from silt to coarse sand.

It is recommended that if the formula is used, an appropriate degree of conservatism be applied to the value of C selected. Depending on the particular circumstances, conservatism may involve selection of a low or high value of C.

(ii) Kozeny-Carman formula. Head (1985) discusses the application of the Kozeny-Carman formula and suggests the form

$$k = \frac{2}{fS^2}\left(\frac{e^3}{1+e}\right) \text{m/sec}$$

where S = specific surface

$$= \frac{6}{\sqrt{d_1/d_2}}$$

d_1, d_2 = maximum and minimum sized particles in mm

e = void ratio

f = angularity factor for rounded grains; 1.25 for subrounded grains; 1.4 for angular grains.

This equation attempts to account for the effect of particle size, void ratio and particle shape or permeability.

It is unlikely that the Kozeny-Carman formula is significantly more accurate than the Hazen formula and its use should be limited to initial estimates of permeability in non critical projects.

6.3.3 *Effects of poor sampling on estimated permeability in the laboratory*

Many alluvial soils are stratified *in situ*, e.g. a sandy gravel/gravelly sand deposit in a river bed will, in fact, consist of beds of fine sand, coarse sand, and gravel.

When sampled in boreholes or test pits, particularly below the water table, there is often a tendency to mix the strata, in the drilling process and/or in the sampling. If these mixed samples are used for laboratory permeability tests, or their permeability is inferred from their particle size distribution, it is likely that the permeability for flow along the strata, ie. essentially horizontal permeability, will be significantly underestimated.

Figure 6.32 gives an example where soils A, B and C are present as distinct layers. Their individual permeability estimated using Hazen's formula are as shown for horizontal flow. Soil

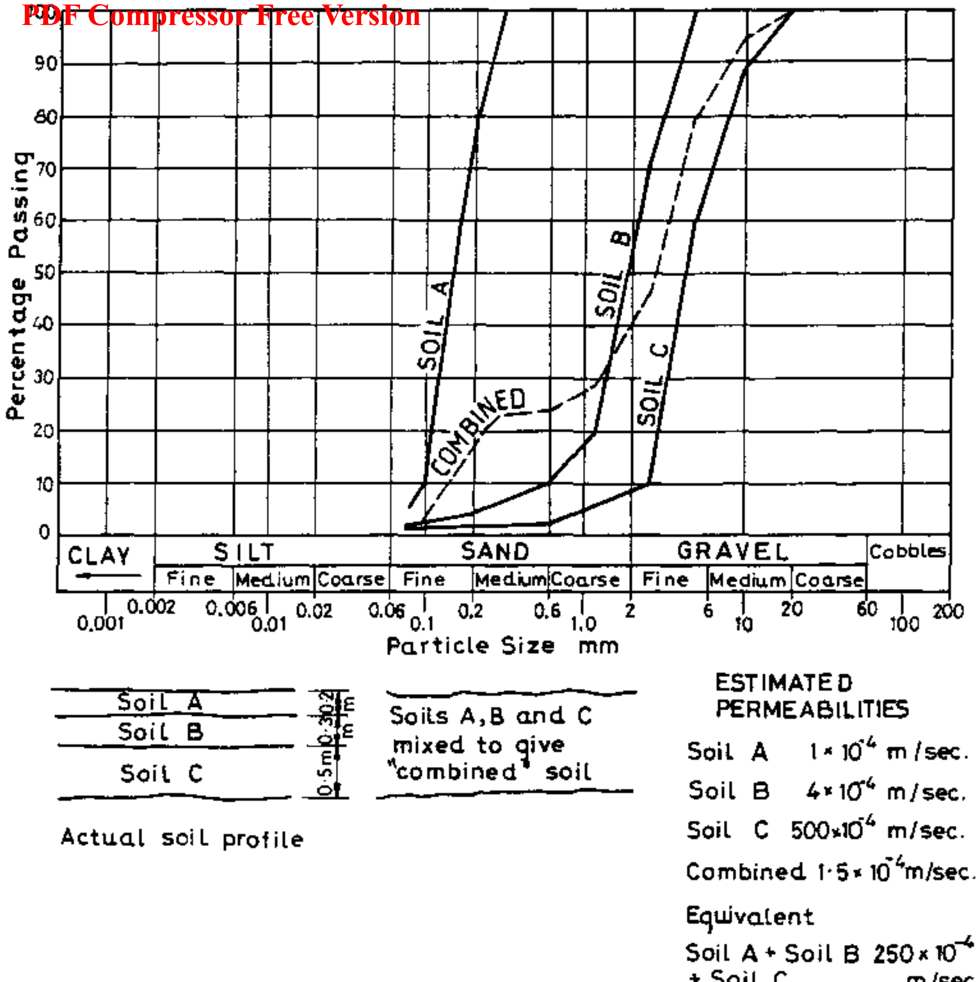


Figure 6.32. Effect of mixing of soil on estimated permeability.

C (gravel) dominates the conditions, and the equivalent horizontal permeability is of the order of 250×10^{-4} m/sec.

If in the drilling and sampling process the soils are mixed in proportion to their thicknesses, the combined soil particle size distribution would be as shown. For the combined soil the permeability is controlled by the finer particles, resulting in an estimated permeability of 1.5×10^{-4} m/sec, or only 1.6% of the actual field condition. This can only be overcome by very careful drilling and sampling, or preferably by in-situ permability testing.

CHAPTER 7

Design, specification and construction of filters

7.1 BASIC REQUIREMENTS FOR FILTERS

7.1.1 'Critical' and 'non critical' filters

As discussed in Chapter 2, filters are provided in earthfill and earth and rockfill dams to control erosion of soil in the dam embankment and from the dam foundations under the forces generated by seepage through and under the dam. These filters are essential to the performance of the dam. Their failure to perform satisfactorily, can lead to piping failure of the dam. Such filters are designated 'critical filters' and the bulk of the discussion in this chapter relates to the design, specification and construction of such filters.

Filter materials are sometimes also provided upstream of the earthfill core of an earth and rockfill dam, and beneath rip-rap in earthfill dams. In many cases these filters are designed to lesser standards in recognition of the less severe erosional forces involved, and the lower risk that the dam would fail if the filters do not perform completely satisfactorily. These filters are termed 'non critical filters' and are discussed in Section 7.2.6.

7.1.2 *General concepts of filter design*

The basic requirements of filters are that they are:

– sufficiently fine grained to prevent erosion of the soil they are protecting,

– sufficiently permeable to allow drainage of seepage water.

Extensive laboratory testing programs, beginning with work by Bertram (1940) who worked under the direction of Terzaghi and Casagrande, followed by the work of the United States Bureau of Reclamation (USBR) (reported in USBR 1955) and US Corps of Engineers (reported in US Corps of Engineers 1941), showed that the basic requirements could best be met by designing the particle size of sand and gravel filters in relation to the particle size of the soil being protected.

The USBR method which has been modified from time to time, e.g. USBR (1977), is described in Section 7.2.1 and has been used widely and successfully for design of filters. More recently further experiments have been carried out by the United States Soil Conservation Service (reported in Sherard et al. 1984a, b, Sherard & Dunnigan 1985) and by Kenney & Lau (1985). These experiments led to recommended modifications of the design rules suggested by USBR (1977) and are described in Sections 7.2.2 and 7.2.3.

An important concept in relation to design of filters was discussed by Sherard & Dunnigan

(1985). They pointed out that 'since about 1970-1975 it has been fairly widely accepted that for important dams, multiple lines of defense' should be provided against leaks in cracks. This idea was applicable especially for high dams in steep walled valleys or other situations where large differential settlement was inevitable. The design measures making up these lines of defense fall into two main categories:

1) Methods to reduce differential settlement and the likelihood of a concentrated leak. (a) Excavation of rock abutments to achieve flatter slopes and reduce abrupt changes in slope. (b) For central core dams, using an upstream sloping impervious core instead of a vertical core. (c) Compacting the bottom half of the impervious earth core at a relatively lower water content and to as high density as practicable, to reduce its compressibility. (d) For the upper part of the impervious core, compacting at a higher water content, to make it flexible and capable of following imposed strains without cracking. (e) Using plastic clay instead of silty sand for impervious core, also because of flexibility. (f) Using a strip of impervious core at especially high water content at the abutment contacts; sometimes using also a more plastic clay for this strip. This strip is used to obtain a good seal between the soft, relatively erosion resistant soil and the rock, and to allow the embankment to settle with respect to the rock abutment, without cracking. (g) Arching embankment dams in plan, to increase the longitudinal stress when water pressure pushes the dam crest downstream.

2) Measures to control erosion by water flows concentrated in cracks that develop in spite of preventive measures. (a) Use of conservative downstream filters (transitions) to prevent such flows from carrying eroded soil out of the core. (b) Use of plastic clay for impervious core, which is believed to be more resistant to erosion by concentrated leaks. (c) Use of sand in zones upstream of the core. The sand would be washed into and fill the leakage channels.

Sherard (1985) shows that hydraulic fracture commonly leads to concentrated leaks in embankment dams, even though such fracture may not be readily observed and is not restricted to dams which are subject to unusually large differential settlement. Sherard & Dunnigan (1985) then suggested that the emphasis on controlling erosion should not be on the dam core (as in the 'multiple lines of defense' approach) but on the filters, they indicated:

'In the past practice the designer held as an axiom that:

The impervious core is the most important element in the dam. As long as the impervious core remains intact, with no cracks or other concentrated leaks, the dam will be safe. Therefore, the primary and most important objective of the design is to provide measures which will minimize the likelihood of a concentrated leak to the greatest extent possible.

Based on the current available experience the designer is now inclined to see the situation differently: We have been deluded in the past thinking that the impervious sections of our dams remain intact. Evidence now shows that concentrated leaks commonly develop in well designed and constructed dams. It is now clear that the most important element in the dam is the filter (or transition zone) downstream of the core. By providing a conservative downstream filter, we can quit worrying about possible concentrated leaks through the core.'(Sherard & Dunnigan 1985).

The current authors agree with this philosophy and strongly recommend careful design of filters. As discussed in Section 7.2.5, it is recommended this be based on the work of Sherard & Dunnigan (1985), with some additional controls based on USBR and US Corps of Engineers and the authors' own experience to assist in determination of the overall grading of the filters.

7.2 FILTER DESIGN METHODS

7.2.1 *USBR method*

The USBR method as described in USBR (1977) is:

(i) a) D_{15F}/D_{15B} = 5 to 40, provided that b) the filter does not contain more than 5% passing 75 μm. The fines should be cohesionless.

(ii) $D_{15F}/D_{85B} \leq 5$

(iii) The grain size curve of the filter should be roughly parallel to that of the base material

(iv) Maximum size particles in filter = 75 mm to prevent segregation during placement

(v) For base materials which include gravel particles, the base material D_{15F}, D_{85B} etc should be analysed on the basis of the gradation of the soil finer than 4.76 mm. This is in recognition that broadly graded clay-sand-gravel mixes, may not 'self filter,' and the clay may erode through a filter designed on the overall grading.

In the above

– D_{15} is that particle size at which 15% of the soil particles are smaller.

– B indicates 'base' material (the soil to be protected from erosion).

– F indicates filter.

Criterion (i a) and (i b), i.e. $D_{15F}/D_{15B} \geq 5$ ensures that the filter is more permeable than the base soil. Criteria. (ii) and (iii) are designed to ensure that the filter is sufficiently fine to control erosion of the base.

The authors' experience with applying this method is that while it can be successfully applied to base soils which are clayey sands or sandy clays, it cannot be rigorously applied to base soils with a high clay and silt content. In particular:

– rule (iii) cannot be followed (this is also the experience of Sherard & Dunnigan 1985),

– rule (i b) will almost always overrule (i a), often to the concern of inexperienced engineers, who feel that the 'mathematic rule,' i.e. (i a) should stand.

The authors' approach has been to be comfortable with rule (i b) overriding rule (i a), to ignore rule (iii) and substitute in its place the US Corps of Engineers (1941) requirement that the uniformity coefficient $D_{60F}/D_{10F} \leq 20$.

7.2.2 *USSCS (Sherard & Dunnigan) method*

The USSCS (United States Soil Conservation Service) (Sherard et al. 1984a, b and Sherard & Dunnigan 1985), carried out extensive laboratory testing to check filter criteria. They used several different test apparatus to simulate a concentrated leak in a dam.

7.2.2.1 *For 'base' soils in the fine to coarse sand range (D_{15B} = 75 to 2.36 mm)*

Figure 7.1 shows the apparatus used to test these materials.

Tests were carried out on a range of single sized base soils by rapidly applying 400 kPa pressure to the top of the sample, catching the water (and any eroded base material) in a bucket below after 30 to 60 seconds. If there was no sediment, the whole apparatus was vibrated on a vibrating table and sediment (if any) collected. A filter was:

– Successful: if no base was eroded;

– Failure : if significant erosion of base occurred in first 60 seconds and continued at the same rate;

– Borderline: if erosion only occurred under vibration.

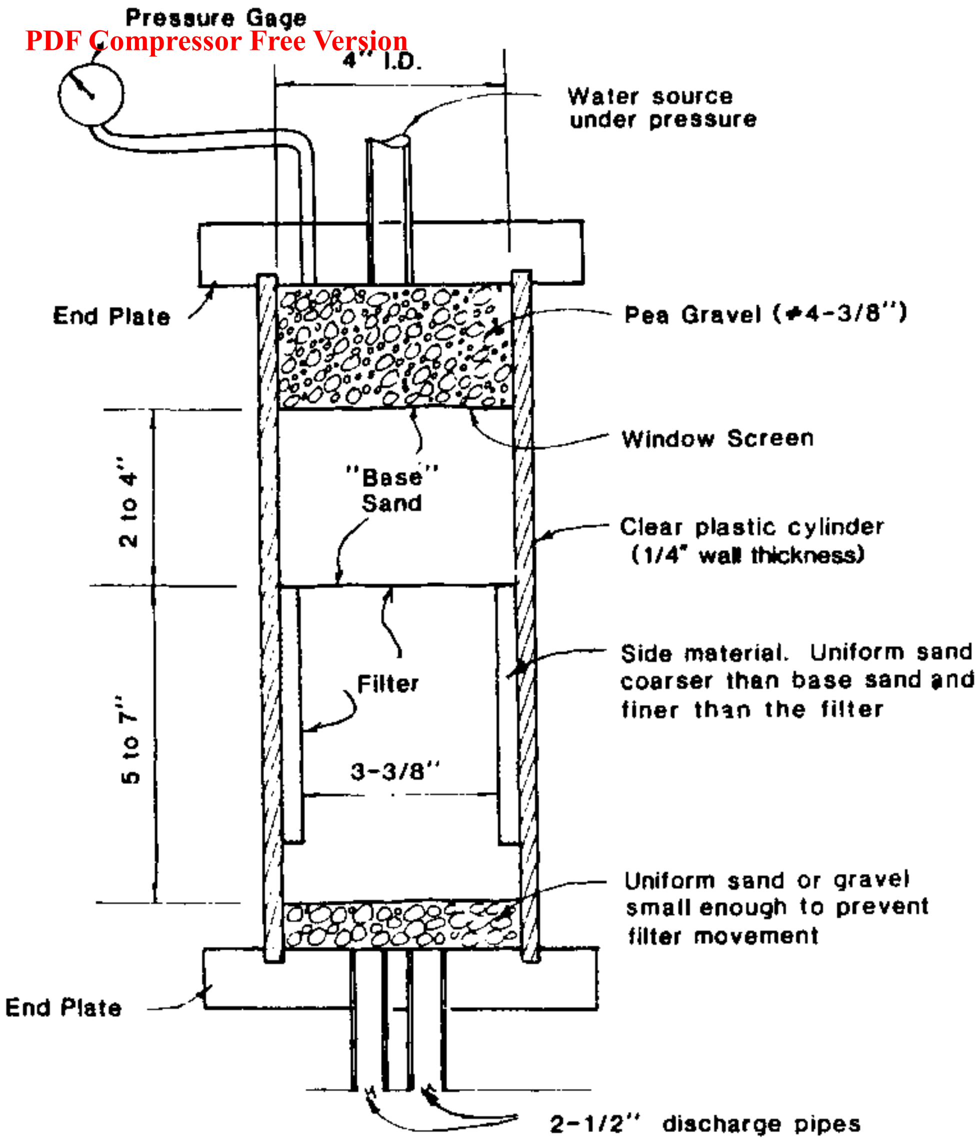


Figure 7.1. USSCS filter test apparatus details for fine to coarse sand base soils (Sherard et al. 1984a).

As is apparent from Figure 7.2, this gave very consistent results, indicating that erosion of the base soil will not occur provided that $D_{15F}/D_{85B} \leq 9$.

Sherard et al. (1984a) also showed that the original USBR (1955) experiments on which $D_{15F}/D_{85B} \leq 5$ was based, had been conservatively interpreted in that, while some initial washing of base material occurred, this ceased, as the base 'self filtered' on the filter. These results were consistent with the Bertram (1940) experiments.

They showed that criteria based on D_{15F}/D_{15B} or D_{50F}/D_{50B} ratios did not give consistent results and they recommended they not be used. They also disputed the $D_{60F}/D_{10F} \ngtr 20$ rule,

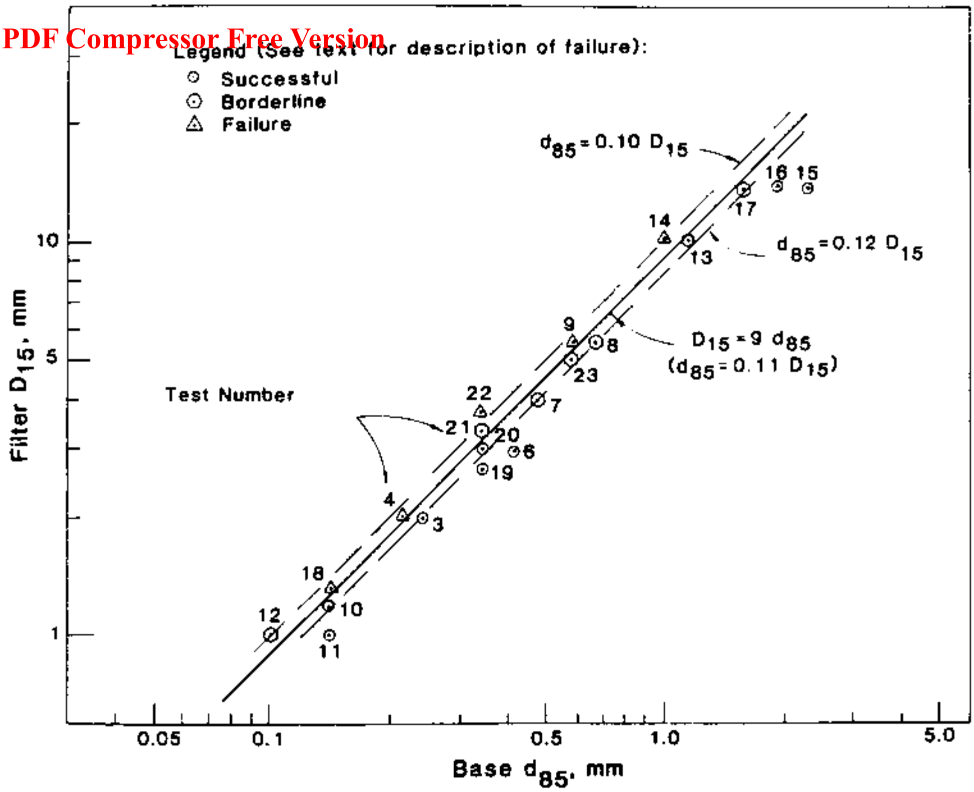


Figure 7.2. Relationship between D_{15F} and D_{85B} in USSCS filter tests on fine to coarse sand base soils (Sherard et al. 1984a).

indicating that segregation problems were caused by particles coarser than 50 mm rather than the grading uniformity.

7.2.2.2 *For silt and clay base soils*

As discussed earlier, concentrated leaks are likely to occur due to cracking of the earthfill core by differential settlement, hydraulic fracture or other means. Sherard et al. (1984a) suggest that this can lead to locally very high gradients adjacent to the filter, as shown diagrammatically in Figure 7.3.

They decided that this condition was not simulated by the test arrangement shown in Figure 7.1. They, therefore, simulated the crack by preforming a slot in the base soil sample as shown in Figure 7.4.

They also carried out other tests by making a slurry of the base soil as shown in Figure 7.5.

As for the tests for fine to coarse sand in the 'slot' and 'slurry' tests, soils were subject abruptly to 400 kPa pressure. This often resulted in an initial surge of erosion, but with successful filters, the water became clear as the base soil self filtered on the filter. They found that testing at lower pressures (14 kPa) did not give consistent results.

A wide range of soils were tested, including dispersive clays, giving the results shown in Figure 7.6.

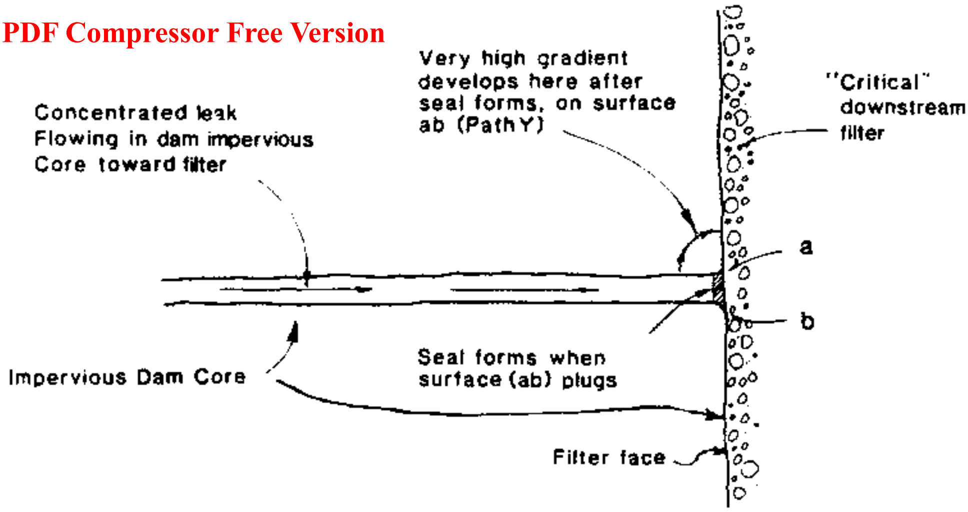


Figure 7.3. Sketch showing concentrated leak through dam core discharging into downstream filter (no scale) (Sherard et al. 1984b).

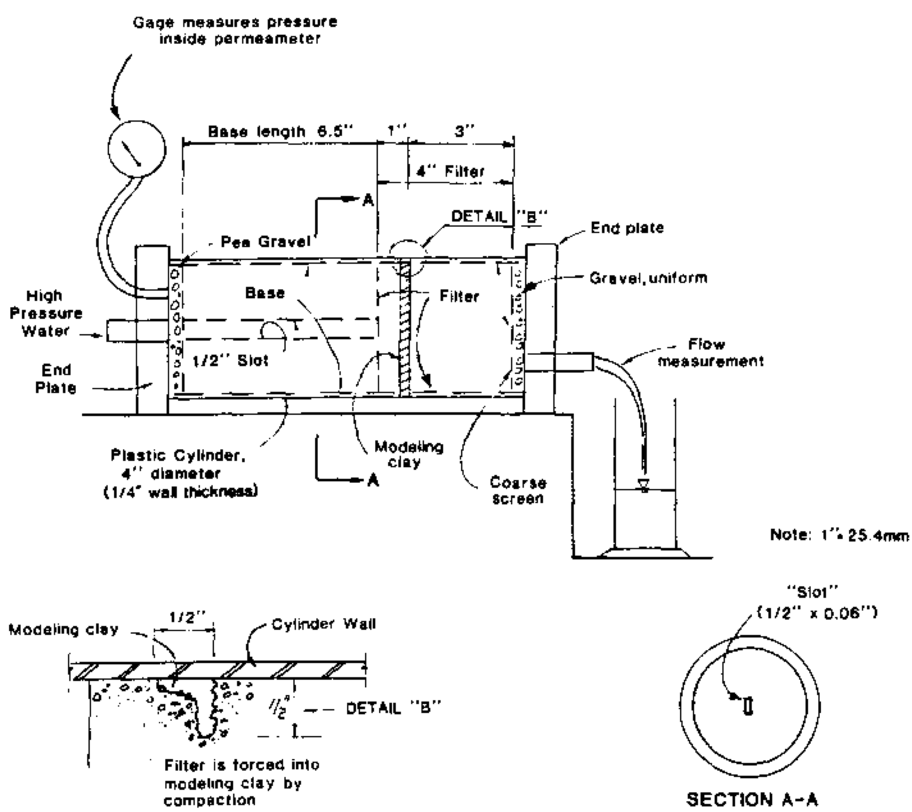


Figure 7.4. USSCS high pressure 'slot' test apparatus details (schematic, no scale) (Sherard et al. 1984b).

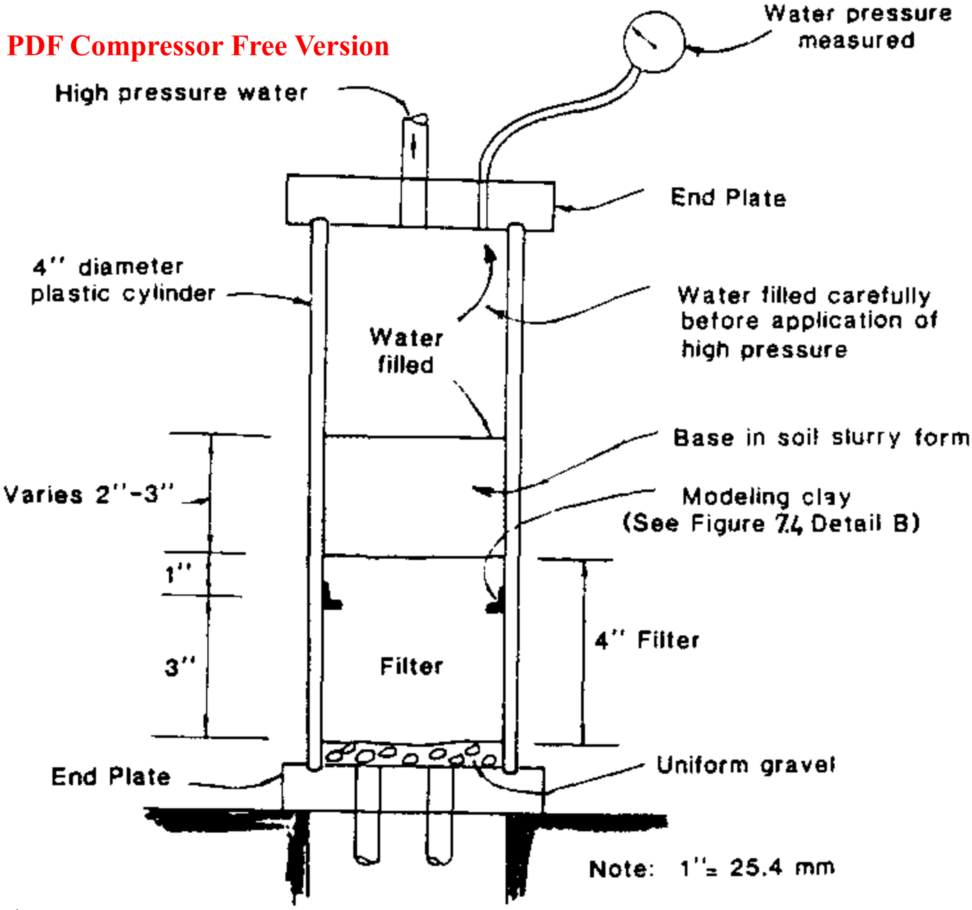


Figure 7.5. USSCS high pressure 'slurry' test apparatus details (schematic, no scale) (Sherard et al. 1984b).

The 'slot' and 'slurry' type tests gave identical results, and as for the tests on fine to coarse sand base soils, D_{15F}/D_{15B} and D_{50F}/D_{50B} relationships were found to be inconsistent or impracticable and Sherard and Dunnigan recommended that they should not be used.

7.2.2.3 *USSCS no erosion filter test*

In the tests described in Sections 7.2.2.1 and 7.2.2.2, a 'successful' filter allowed a small amount of erosion and penetration of fines into the filter, before the filter sealed.

A 'no-erosion' success criterion was applied in a later series of tests by Sherard & Dunnigan (1985). A slightly different test layout was adopted as shown in Figure 7.7.

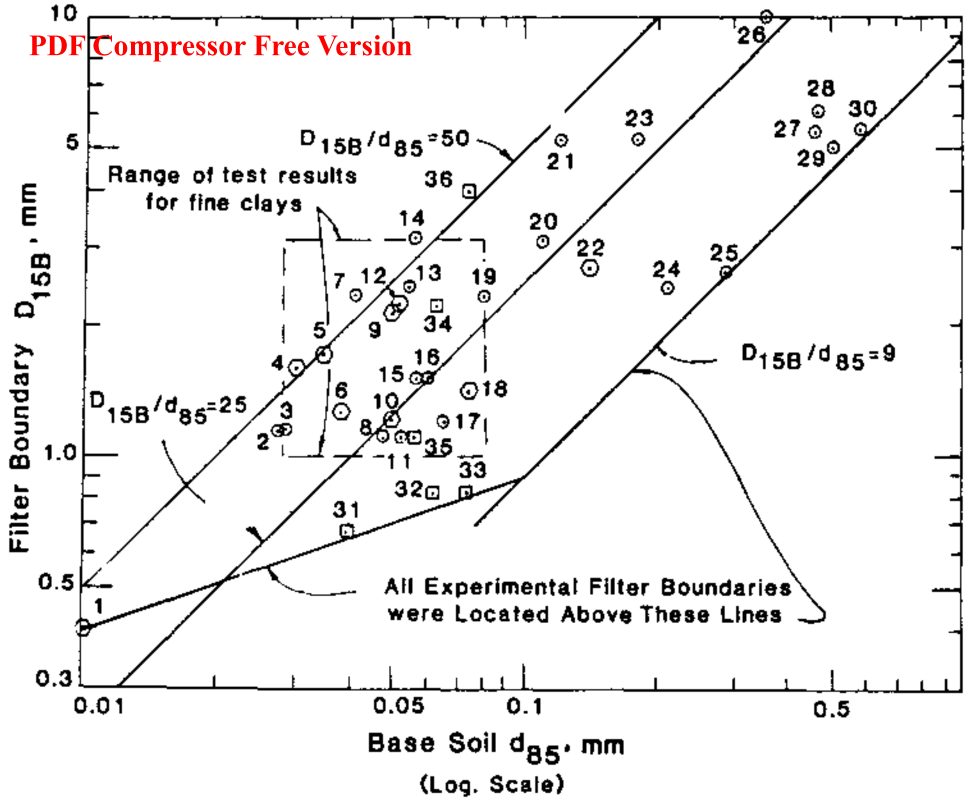
In these tests:

– The sample is subject to 400 kPa pressure rapidly (tap water for non dispersive clays, distilled water for dispersive clays),

– Observed for 5 to 10 minutes – measuring flow and turbidity,

– Shut off, dismantle and observe erosion of base soil.

For a filter to be considered successful there should be:



⊙, ⊙ and ⊡ Represent experimentally determined boundaries between filter success and failure

⊙ Tests on nondispersive clay ⊡ Tests on silts (ML and CL-ML)

⊙ Tests on dispersive clay ⊙20 Test on Soil No. 20

Figure 7.6. Results of USSCS 'slot' and 'slurry' tests on silt and clay base soils (Sherard et al. 1984b).

– No visible increase in hole diameter in base soil,

– very slight erosion of base soil which seals the filter-quantity too small to see.

For 'unsuccessful' filters:

– The hole is eroded progressively larger.

For 'just unsuccessful' filters:

– Filter seals rapidly with only a few grams of base soil eroded to seal the filter face.

These tests gave consistently reproducible results and defined a relatively narrow boundary between successful and unsuccessful filters. The filter boundary D_{15B} defined for silts and clays is finer than that defined by the 'slot' and 'slurry' tests, but still gives reasonable filter sizes for direct use in design, without the need for slot testing of the soil being assessed (as was recommended in Sherard & Dunnigan (1984) for clays with little or no sand).

The test results were also found to be independent of whether soils are dispersive or non dispersive. The flow rate in the hole is large enough to erode all soil types.

Arising from these tests, Sherard & Dunnigan (1985) recommended:

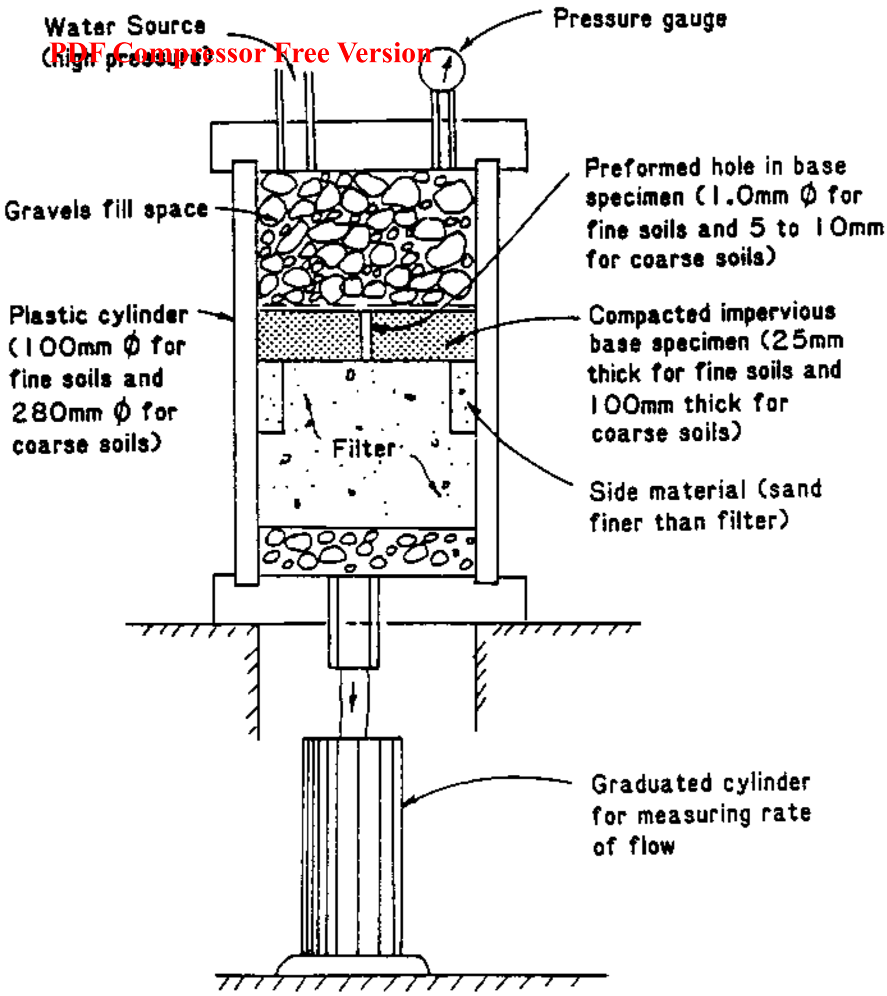


Figure 7.7. USSCS 'no erosion' filter test apparatus (schematic, no scale) (Sherard & Dunnigan 1985).

1. For all soils with a gravel component (except Group 3 below), the filters should be designed on the grading of that part of the soil finer than 4.76 mm.

2. Impervious Soil Group 1 (fine silts and clays): For fine silts and clays that have more than 85% by weight of particles finer than the 75 μm sieve, the allowable filter for design should have $D_{15F} \leq 9\,D_{85B}$.

3. Impervious Soil Group 2 (sandy silts and clays and silty and clayey sands): For sandy (and gravelly) impervious soils with 40 to 85% by weight (of the portion finer than the 4.76 mm sieve) finer than the 75 μm sieve, the allowable filter for design should have $D_{15F} \leq 0.7$ mm.

4. Impervious Soil Group 3 (sands and sandy gravels with small content of fines): For silty and clayey sands and gravels with 15% or less by weight (of the portion finer than the 4.76 mm sieve) finer than the 75 μm sieve the allowable filter for design should have $D_{15F} \leq 4\,D_{85B}$ where D_{85B} can be the 85% finer size of the entire material including gravels.

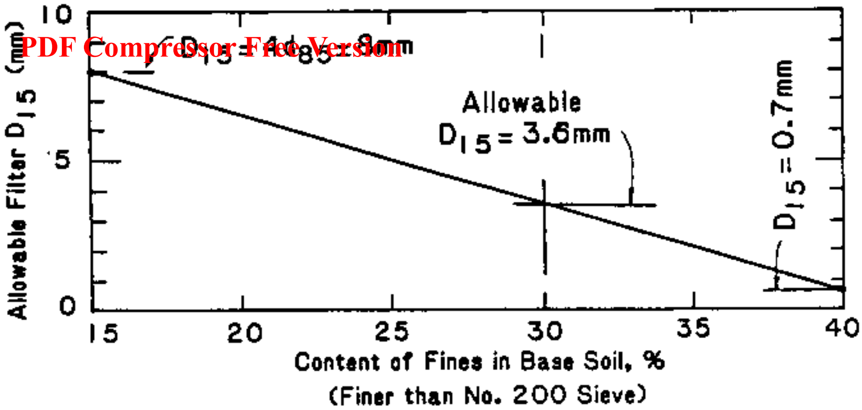


Figure 7.8. Determination of allowable downstream filter for impervious Soil Group 4 (having between 15 and 40% finer than 75 μm sieve) (Reproduced from Sherard & Dunnigan 1985).

5. Impervious Soil Group 4: For coarse impervious soils intermediate between Groups 2 and 3 above, with 15 to 40% passing the 75 μm sieve, the allowable filter for design is intermediate, inversely related linearly with the fines content and can be computed by straight line interpolation. As an example (Fig. 7.8) for an impervious sandy soil with 30% of silty or clayey fines and $D_{85B} = 2$ mm, the allowable filter for design is inbetween the value of $D_{15F} = 0.7$ mm (for soils of Group 2) and $D_{15F} = 4 \times (2) = 8$ mm (for soils of Group 3), and is calculated as follows:

$$D_{15F} = \frac{40 - 30}{40 - 15}(8 - 0.7) + 0.7 = 3.6 \text{ mm}$$

6. As well as having D_{15F} sizes as set out above, the filters for Soil Groups 1 and 2 must be composed wholly of sand or gravelly sand in which $\not>$ 60% is coarser than 4.76 mm and the maximum particle size is 50 mm.

7. The above criteria can be applied for all soils in Groups 1 and 2 regardless of the shape of the particle size distribution curve. For soils of Groups 3 and 4 the criteria apply to reasonably well-graded soils: for soils in Groups 3 and 4 which are highly gap-graded it is desirable to provide a filter for the finer portion of the gap-graded soil, or carry out No Erosion Filter Tests in the laboratory to select the appropriate filter.

Sherard & Dunnigan (1985) indicate that these criteria already incorporate an adequate factor of safety.

De Mello (1989) discusses the Sherard and Dunnigan tests, and suggests that they can be plotted to show that the recommended criteria still have a significant probability of failure, if one allows for statistically reasonable segregation in the filter as placed. There would seem to be some validity in the argument but against this, is the good practical experience when such criteria are used.

The criteria listed above do not include specific reference to limiting the 'fines' content, i.e. silt and clay passing 75 μm sieve, except that (6) indicates filters for Soil Groups 1 and 2 should be composed 'wholly of sand and gravel.' Some limitation on fines content and nature such as that required by the USBR (1977) seems to be implied ie. filters should contain not more than 5% fines passing 75 μm, and the fines should be 'cohesionless.'

It should be noted that for Soil Group 1, hydrometer particle size analysis will be required to define the particle size below 75 μm.

7.2.3 *Research by Kenney, University of Toronto*

Kenney and co-workers at the University of Toronto, Canada, have carried out extensive testing on filters. Their work is reported in Kenney et al. (1985) and Kenney & Lau (1985).

7.2.3.1 *Filters for cohesionless base soils*

Kenney et al. (1985) developed the concept of a controlling constriction size D_C*, which is a size characteristic of the void network in a granular filter, and is equal to the diameter of the largest particle that can possibly be transported through the filter by seepage. They show that:

$$D_C* \leq 0.25\, D_{5F}$$

and

$$D_C* \leq 0.20\, D_{15F}$$

They found that D_C* is primarily dependent on the size of the small particles in the filter, and not strongly dependent on the shapes of the particle size distribution curve for the filter as a whole.

The test setup used by Kenney et al. (1985) is similar to the USSCS apparatus shown in Figure 7.1, but the seepage gradient was between 3 and 50. The cell was tapped lightly during the test to induce vibration. As might be expected this was found to have a significant effect on the results with vibration dislodging particles and facilitating their penetration into the filter.

Kenney et al. (1985) suggest that for filtering cohesionless base soils the following relationships should be applied

$$D_{5F} < 4\, D_{50B}$$

and

$$D_{15F} < 5\, D_{50B}$$

This is based on the requirement that the cohesionless bases and filters have a uniformity coefficient $Cu \leq 6$, and the coarser of the filters given by the relationships should be adopted. If the base soil has a potential for 'grading instability,' i.e. it will not self filter, Kenney et al. suggest that consideration of allowable loss of fines may be necessary.

Kenney & Lau (1985) discuss experiments which seek to assess grading requirements for soils which will or will not be subject to grading instability. They tested the range of cohesionless soils shown in Figure 7.9.

They proposed a method of describing the shape of a grading curve which allows assessment of whether the soil will be subject to grading instability. This method is shown in Figure 7.10.

In the graphs F is the fraction of soil finer than the point D on the grading curve, and H is the mass fraction of the soil measured between particles D and 4D.

Soils are potentially unstable, if over the range $F = 0$ to 0.2 for soils with $Cu > 3$, and $F = 0$ to 0.3 for soils with $Cu < 3$, the shape curve falls below (to the left) of the boundary line.

Wolski (1988) concludes that soils with a uniformity coefficient of $Cu < 20$ (as proposed by the Corps of Engineers) would fall within the stable zone of Kenney & Lau's (1985) experiments.

Kenney & Lau (1985) and Wolski (1988) point out that soils which fall within the 'unstable' region using the laboratory tests are only potentially unstable, and cite cases in which filters have apparently performed satisfactorily even though laboratory tests show they are in the

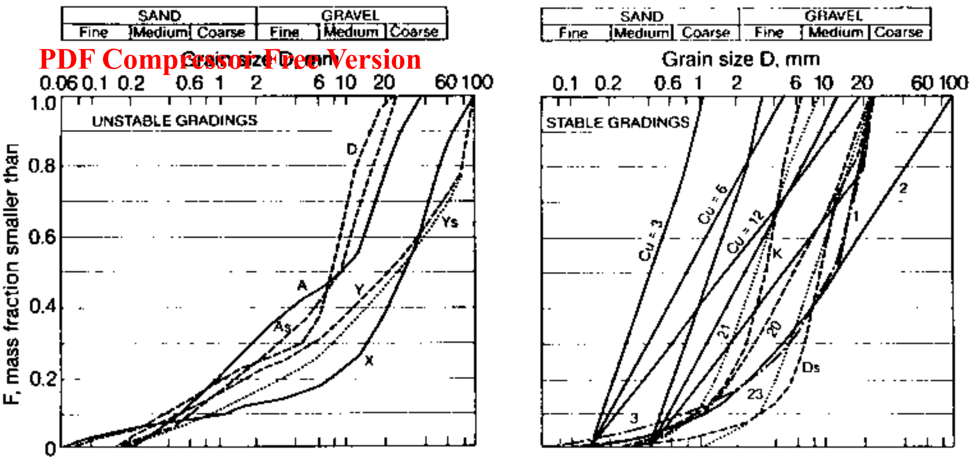


Figure 7.9. Soils tested by Kenney & Lau (1985) to assess grading instability.

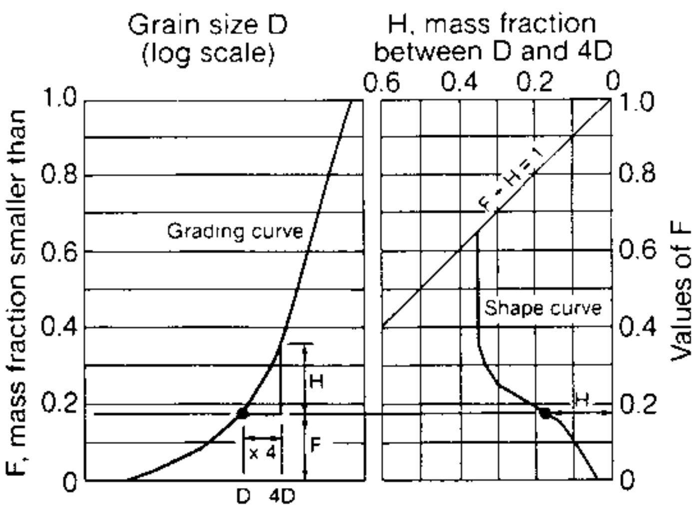


Figure 7.10. Kenney & Lau (1985) shape curve method.

unstable region. Both suggest the best way to check for potential instability is to carry out experiments on the soil in question.

7.2.3.2 *Filters for cohesive soils*

Kenney et al.'s (1985) work was largely on cohesionless base soils. They describe personal experience of two dam cores composed of widely graded glacial tills with particles from gravel to clay size, where silt and clay size particles (finer than 60 μm) were removed through the filters by seepage. They attribute this to low seepage velocities being unable to transport the coarser sand particles in the soil to form a 'self filter.' They suggest a rather arbitary selection of particle size in the base soil (20 μm in the example given) which must not pass through the filter (ie. this becomes the controlling constriction size D_C*), and application of $D_C^* \leq 0.25\ D_{5F}$ and $D_C^* \leq 0.20\ D_{15F}$.

They point out this gives finer filter requirements than those given by Sherard et al. (1984b) and coarser than those of the method proposed by Vaughan & Soares (1982). They concluded that to design on use of precedents and experiments (essentially the method used by Sherard et al. (1984b) is acceptable, but that checking with some other approach may be prudent.

7.2.4 *'Perfect' filter method – Vaughan and Soares*

Vaughan & Soares (1982) provoked considerable discussion when in a review of partial piping failure of Balderhead Dam in northern England, they claimed that in some base soils self filtering could not be relied upon, and that it was necessary to design filters to prevent the passing of clay flocs into the filter.

The core for the Balderhead Dam had been constructed from well graded glacial till, which had a high resistance to erosion. Filters were crusher run hard limestone with grading limits shown on Figure 7.11.

Cracks occurred in the dam core by hydraulic fracture, and Vaughan & Soares (1982) postulated that the partial piping failure which occurred more than a year after the dam was filled was due to erosion of the finer clay particles in the cracks. The flow velocity in the cracks was postulated to be too low to transport coarser particles from the core, so preventing self filtering within the core material, and allowing the clay particles to erode into the filter. As discussed in Section 7.2.2, Kenney et al. (1985) observed similar behaviour in two dams constructed of widely graded glacial till.

To overcome this, Vaughan & Soares (1982) proposed the concept of a 'perfect filter.' In this

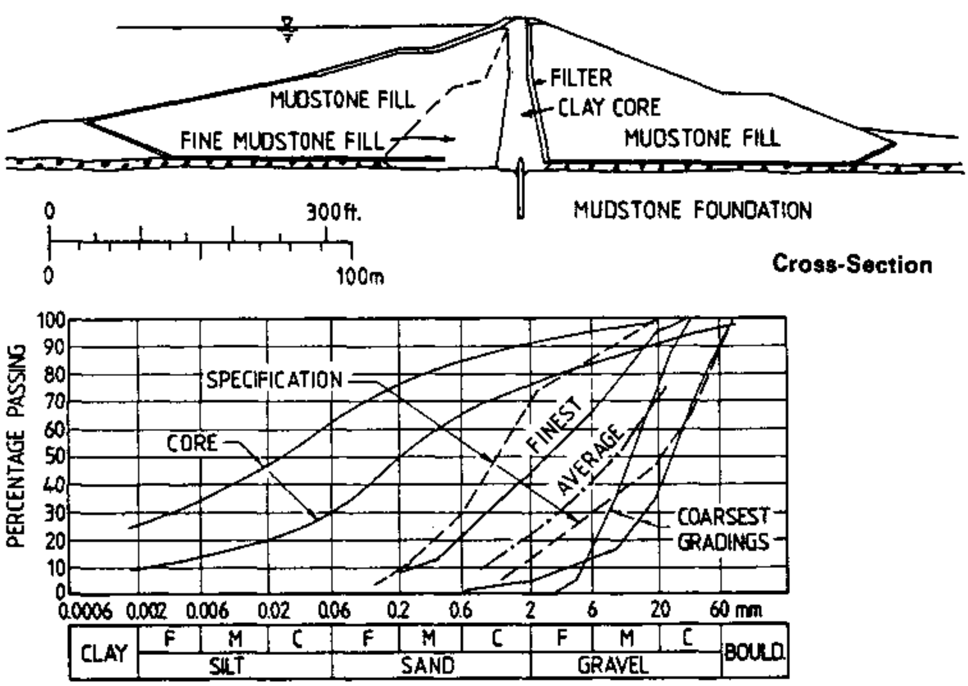


Figure 7.11. Balderhead Dam core and filter gradings (Vaughan & Soares 1982).

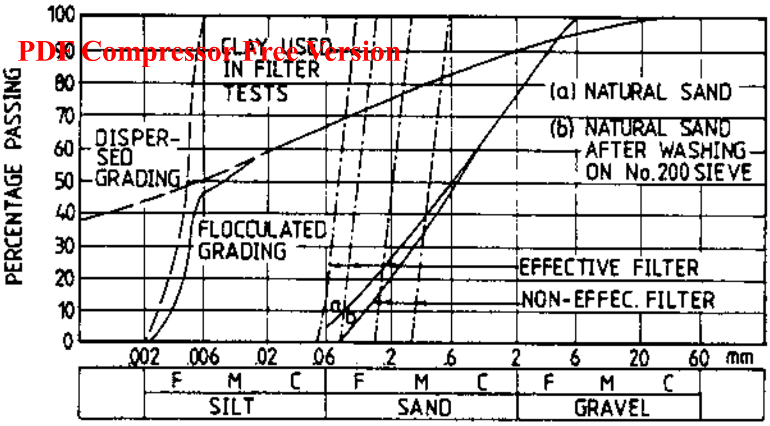


Figure 7.12. Filter designs for Cow Green Dam (Vaughan & Soares 1982).

approach, the particle size distribution of the clay core is obtained in a hydrometer test, but with no dispersant added. The water used for the test should be of the same chemistry as the water to be seeping through the dam since the floc size (degree of dispersion) is dependent on the water chemistry.

Having determined the particle size distribution, a sample of the floc sized sediment is prepared by sedimentation, and used in filter experiments. In these experiments the flocs are introduced to the filter in a dilute suspension.

Tests carried out in this manner on soils from two dams indicated the need for a significant silt fraction in the filter, and that even a fine to medium sand size filter may not be satisfactory, e.g. for Cow Green Dam shown in Figure 7.12.

The general tone of the discussion which followed the paper was that based 'on experience with other laboratory experiments and performance of dams' the Vaughan and Soares method was too conservative. Vaughan and Soares in the closure discussion conclude 'the writers would stress that the need to meet this criteria completely is one for the designer of any particular dam to assess, along with other factors involved in the provision of effective filters.'

The authors are not aware of any general acceptance of the 'perfect filter' concept. Vaughan and Soares' case is weakened somewhat by the coarse grading of some of the filters as constructed for Balderhead Dam – these would not meet USBR (1977) or Sherard & Dunnigan (1985) criteria. However it does seem appropriate that for widely graded soils, particularly those of glacial origin, consideration be given to adopting conservative filter designs, possibly checking the implications of using the 'perfect filter' concept.

7.2.5 *Recommended design method*

It is recommended the following approach be adopted for design of critical filters:

1. Use Sherard & Dunnigan (1985) as outlined in Section 7.2.2, except that for all base soils (i.e. including Group 3 soils), the filter sizing should be based on the grading of that part of the soil finer than 4.76 mm. As required by rule 7 in Sherard and Dunnigan, when the soils are highly gap graded, the filter should be designed on the finer portion of the gap graded soil.

2. The filters should not contain more than 5% fines passing 75 μm, and the fines should be

non plastic. Where high permeability is required, not more than 2% fines passing 75μm should be allowed. This would be particularly important for vertical and horizontal drains.

3. The uniformity coefficient D_{60F}/D_{10F} should not exceed 20 (D_{60F} on coarse limit of filter, D_{10F} on fine limit of filter.

4. For major projects, particularly those involving dispersive soils, no-erosion filter tests as described by Sherard et al. (1984b) should be carried out using water with the same chemistry as the expected seepage water.

5. For widely graded base soils of glacial origin (similar to those at Balderhead Dam) the design should be checked using the 'perfect filter' concept and that design adopted if practicable.

7.2.6 *Particle size for non critical filters*

The filter Zone 2C, upstream of a central core earth and rockfill dam is not subject to continuous seepage exit gradients or to the risk of high exit gradients if the core cracks.

Zone 2C filters can therefore be designed much less conservatively than Zone 2A and 2B filters. It is common only to require that 'Zone 2C filters shall be constructed of well graded rockfill with a maximum size of 150 mm.' This will result in most cases in a sand/gravel/cobble size mixture of rock fragments, which provides a transition between the earthfill core and the upstream rockfill. Since this zone will be won from the quarry (probably after passing through a 'grizzly' to remove oversize rock) it is relatively inexpensive, and it would be normal to place at least 4 m width, possibly 6 m on a larger dam.

A similar filter may be used under rip-rap. The riprap is being constructed as a wide zone of dumped rock (rather than a thin layer of graded rocks) and wave action is not severe, and the earthfill is not dispersive.

7.3 SPECIFICATION OF SIZE AND DURABILITY OF FILTERS

7.3.1 *Particle size distribution*

The design rules outlined in Section 7.2 do not in themselves entirely allow selection of the particle size distribution of filters. The emphasis in those rules is on 'spot' limitations within the grading curve. For filters to be manufactured and placed in a dam the full particle size distribution limits have to be designed, as for example in Figure 7.13.

Some factors which should be taken into account are:

– The wider the particle size grading allowed, and the more compatible this is with the available sand and gravel, the lower the cost of manufacture. If natural sand and gravel deposits are available it may be possible to match the filter grading requirements to the grading of those deposits.

– Breakdown of particles occurs during placement (particularly coarser particles) and compaction. Specifications should apply to the filter after placement in the dam. This means that the contractor may have to manufacture the filters coarser than the specifications.

– On most dam projects, concrete aggregates and filters are produced in the same crushing and screening plant, so sieve sizes on which specifications are based should be the same for both.

– The design rules recommended for use in Section 7.2 are largely biased towards design of

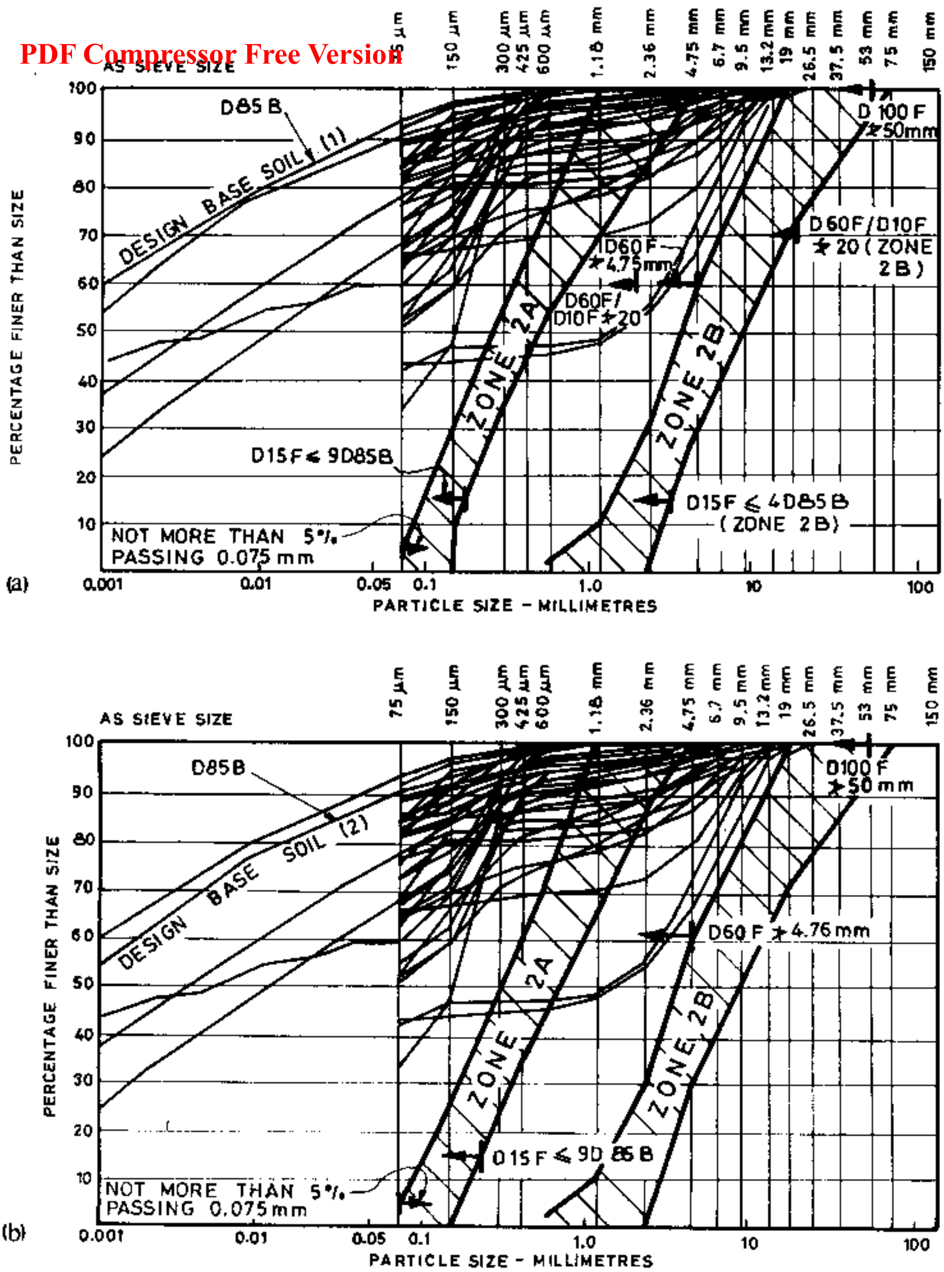


Figure 7.13. Example of filter design – clay and sandy clay base soil. a) Design based on finest soil in borrow area; b) Design based on marginally coarser soil.

filters to be placed against Zone 1 earthfill in a dam. The following requirements do not apply to design of Zone 2B filters: The requirement to base the design on the passing 4.76 mm fraction of the base soil; the requirement to limit D_{60} of the filter to less than 4.76 mm; the requirement to limit the maximum size to 50 mm.

Charleton & Crane (1980) and Magnusson (1980) discuss some of the practical problems associated with filter manufacture and design.

The following examples highlight some of the design decisions necessary.

Case A: Filters for a clay and sandy clay base soil. Figures 7.13a and b show the plots of particle size distribution for soil samples taken from a borrow area which consists of alluvial clay and sandy clays. The soils are distributed randomly throughout the borrow area. Figure 7.13a shows filters designed using criteria recommended in Section 7.2, based on the finest 'base' soil in the borrow area.

The design in Figure 7.13b is based on a slightly coarser base soil particle size distribution. This might be justified on the grounds that:

– only a small percentage of the soil in the borrow area is finer than this design particle size distribution,

– this soil will be mixed with coarser soil during the borrow, transport and placement operation.

Use of the slightly coarser base soil in Figure 7.13b allows a significantly coarser D_{15} for the filter, and hence wider grading limits for the Zone 2A filter. Zone 2B is not affected.

Whether such an approach is adopted depends on the particular borrow area, method of borrow etc. Whatever approach is adopted, the specification for the base soil (Zone 1) should have a fine limit the same as that used for design of the Zone 2A filter.

In these designs, the grading limits for Zone 2A in Figure 7.13a are quite narrow. This is forced by the fine base soil particle size. These narrow limits would lead to higher costs of production than the rather wider limits in Figure 7.13b.

In this example, the D_{85B} is less than 75 µm, and must be obtained from hydrometer particle size analysis. This should be done using the standard approach, in which, dispersant is added to the soil as part of the test procedure.

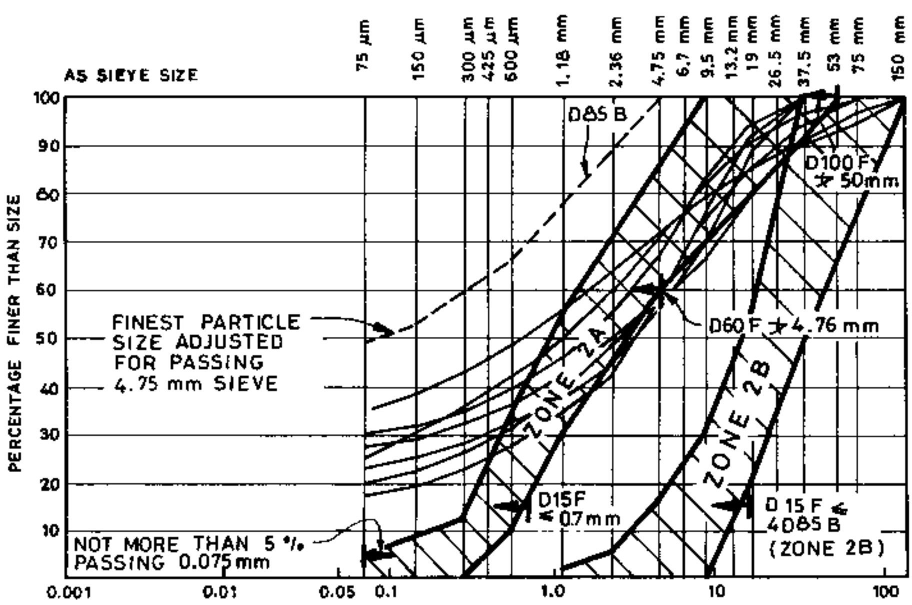


Figure 7.14. Example of filter design – clayey sandy gravel base soil.

For Zone 2B, a maximum particle size of 75 mm has been adopted. Larger size particles may be used in Zone 2B filters but at the risk of segregation in stock piles and during placement.

Case B: Filters for a clayey sandy gravel base soil. Figure 7.14 shows the plots of particle size distribution for soil samples taken from a borrow area which consists of clayey sandy gravel. The soils are distributed randomly throughout the borrow area.

The design in Figure 7.14 is based on the particle size of the base soils adjusted to that portion passing the 4.75 mm sieve. This adjusted 'base soil' is a type 2 soil, so by the recommended design procedure, $D_{15F} \leq 0.7$mm. For the design shown, it has been assumed that it is desirable to have the coarsest filter compatible with the design rules, so D_{60F} (coarse side) = 4.75 mm has been adopted. The D_{10F} (fine side) is then determined by $D_{60F}/D_{10F} \not> 20$. Other grading limits could be selected which would satisfy the basic requirements and the decision would be based on compatibility with available sand and gravel deposits, and the grading requirements for Zone 2B to meet permeability or erosion control into Zone 3A etc.

It will be noted that the grading limits for this example are wider than for the filters shown in Figure 7.13a and b. Such wider limits are desirable as they should facilitate manufacture and thereby reduce costs.

7.3.2 *Durability*

Filters are required to be constructed of sand and gravel which is sufficiently durable so as not to break down excessively during the mechanical action of placement in the dam, under the chemical action of seepage water, or under wetting and drying within the dam.

Durability requirements should be considered on an individual basis for each dam, and are likely to be more rigorous for high hazard dams, and dams with seepage water which is highly acidic or has high salts content (e.g. some tailings dams). The requirements should also reflect the properties of the available materials. There is no point in specifying highly rigorous requirements if none of the available sources will meet the requirements. However, for a very large, high hazard dam, the requirements should not be relaxed.

Table 7.1 shows the specified durability requirements for several dams constructed in Australia. Also shown for comparative purposes are the requirements for fine and coarse concrete aggregates as specified by Standards Association of Australia (1985).

These specifications can be used as a guide to selecting reasonable limits. The following comments are offered:

a) The Los Angeles abrasion and wet strength, wet/dry strength variation largely assess the susceptibility to breakdown under the mechanical action of placement and rolling in the dam. It can be seen that dam specifications are generally less stringent than AS2758.1. If materials test marginally within the specification it would be advisable to carry out field compaction trials of the filters, to observe directly the degree of breakdown under rolling, and the size of the 'broken down' product. It may be practicable to make design changes e.g. wider or thicker filters to accommodate the breakdown.

b) The Sodium Sulphate soundness test assesses the susceptibility to breakdown under wetting and drying, with the added involvement of sodium sulphate in the solution used for soaking. The sodium sulphate penetrates fine cracks in the aggregates, and expands on crystalization in the drying phase breaking apart the aggregate particles.

It is a particularly severe test for most dam applications where high salt contents are not present. This is reflected in the dam specifications in Table 7.1, where the requirements are less severe than for coarse concrete aggregates (similar for fine aggregates).

Table 7.1. Durability requirements for Zone 2A and 2B filters and for concrete aggregates.

Authority and dam	Zone	General statement	Los Angeles abrasion value	Sodium sulphate soundness	Wet strength and wet/dry strength variation	Other requirements
Water Resources Commission of Queensland, Peter Faust Dam (1989)	2A	Uncrushed, non plastic, free draining, clean, hard, durable sand free of organic material				
	2B	Hard, durable dense, fresh to slightly weathered (≯ 30% slightly weathered in + 4.75 mm size)		≯ 15% on minus 4.75 mm wet/dry	≮ 80 kN wet strength, ≯ 35% variation	
Water Authority of Western Australia, Harris Dam (1989)	2A	Hard, durable fresh rock	≯ 45% at 500 revolutions	≯ 14% on minus 0.3 mm	Not specified	Satisfy organic content in AS1141 Section 34
Melbourne and Metropolitan Board of Works, Cardinia Creek Dam (1970)	2A and 2B	Hard, durable, non plastic	≯ 40% at 500 revolutions	≯ 14% on minus 0.3 mm	Not specified	
Snowy Mountains Hydroelectric Authority, Talbingo Dam (1967)	2A	Hard, durable fresh rock	≯ 40% at 500 revolutions	≯ 14% on minus 0.3 mm		
	2B transition	Moderately weathered to fresh rock unaffected by chemical alteration (rhylite, weathered tuff or porphry)				Zone 2B is wide and not strictly comparable to Zone 2B as discussed in this book
AS2758.1, 1985. Aggregates and rock for engineering purposes	Fine aggregates (< 5 mm)			≯ 15% protected and moderate conditions, ≯ 12% severe conditions		Experience shows fine grained aggregates have generally substantial weathering in their natural environment and rarely require durability testing
	Coarse aggregate	Test either Los Angeles and sulphate soundness OR wet strength and wet/dry strength variation	≯ 35% CG, ≯ 25% FG, moderate conditions; ≯ 30% CG, FG not specified, severe conditions	≯ 12% protected, ≯ 9% moderate, ≯ 6% severe	≮ 80 kN wet strength, ≯ 35% wet/dry variation moderate conditions, ≮ 100 kN wet strength, ≯ 25% wet/dry variation severe conditions	Also specify particle density, shape, water absorption

CG = coarse grained rock; FG = fine grained rock

If materials are testing as marginal for this test, it is important to inspect the product of breakdown. In some rocks, the product will be a fine silt and sand, which is likely to affect filter permeability markedly. In others it is simply the breakup of coarse particles into two or three smaller particles, which does not affect permeability greatly.

It is quite common to have different performance in these tests depending on the size fraction being tested. This is particularly the case for naturally occurring aggregates from a river which drains a mixed geological environment where the mineralogy of the individual particles may vary substantially.

The authors' opinion is that for critical tailings dams which are storing water with a high acidity or high salts content, it may be necessary to impose more severe requirements on sulphate soundness than normally accepted for water dams. It may also be necessary to test with the more severe magnesium sulphate if the water in the tailings dam has a high magnesium sulphate content. On the other hand many tailings dams will not be high hazard structures, and less stringent requirements may be adopted.

c) As discussed in Chapters 3 and 6, rocks which contain significant proportions of carbonate minerals may perform satisfactorily in the laboratory tests discussed above, but are probably not suitable for use in filter zones because of the susceptibility of carbonate in the rock particles to solution and redeposition as cement between the particles, to form a cohesive, brittle mass. Such a cemented mass may sustain a crack, preventing the filter from performing satisfactorily. The authors have seen the effects of such recementation of granular materials in several old fills and pavements.

Rocks containing significant amounts of gypsum are highly susceptible to these effects, and should not be used in filter zones.

Rocks containing significant amounts of sulphide minerals are also suspect because these materials may oxidise and produce sulphate minerals which could be deposited as cement between the particles.

Sand sized volcanic ash (pyroclastic) materials have been shown to develop cohesion, in the transition (filter) zones of Matahina Dam (Chapter 3, Section 3.3). The degree to which the cohesion was developed by solution/deposition while in place in the bank or by particle interlock during compaction is not known.

In the light of the above, the authors recommend that as well as the laboratory tests discussed in (a) and (b) above, the following should always be undertaken:

(i) If the proposed material is bedrock, quarried and crushed, or alluvial sand/gravel, the mineral composition of the particles should be determined

(ii) If material is found to contain even small amounts (e.g. 1 to 5%) of carbonate, gypsum, or sulphide minerals, then it should be subjected to further laboratory and field studies to assess its likely performance in the proposed filter zone.

In the field the studies are primarily of the observational type. For all types of materials a search should be made for existing (preferably very old) deposits of the materials e.g. old fills, pavements or natural screes. In the case of alluvial materials the actual deposit in question is also studied.

The deposits are carefully exposed by trenching or pitting. Evidence of any cohesive behaviour is looked for during excavation, and the exposed faces and spoil are examined for any particles which have become cemented together. Evidence of weathering or solution of particles is also looked for.

When judging the suitability or otherwise of a material, from the results of such observational tests, differences between the environments (particularly moisture) in the old deposit and the proposed filter zone must be taken into account.

(iii) Volcanic ash materials should also be checked carefully for long term chemical stability and for any tendency to develop cohesion when compacted. As well as the laboratory and field tests described in (ii) above, field compaction trials are advisable for sand-sized volcanic ash materials.

7.4 DIMENSIONS, PLACEMENT AND COMPACTION OF FILTERS

7.4.1 *Dimension and placement of filter zones*

The width needed to achieve a proper filter against the protected soil is very small, so the width of filter zones is determined by construction requirements.

The selection of the economic dam zoning should account for filter placement requirements, and on the cost of manufacture of the filter materials.

In most dam projects the cost of manufacture of filters is high and there is a need to keep widths to a practical minimum, the exception possibly being on the few occasions when naturally occurring sand and gravel deposits satisfy filter design requirements and are therefore relatively inexpensive.

The following is offered as a guide to practical minimum widths:

a) Filters upstream or downstream of an earth core, when constructed by end-dumping off a truck should be at least 2.5 and preferably 3 m wide.

b) If a spreader box such as that shown in Figure 7.15 is used, a minimum width of 1.5 m is practicable. The filter material is dumped off the truck into the spreader box, which spreads the filter out of its base as it is pulled along by a small bulldozer.

c) If filter materials are very scarce or high cost, formwork can be used to contain bands of filters as narrow as one metre. Sherard et al. (1963) show an example of such placement. This is very unusual and would only be contemplated in exceptional circumstances.

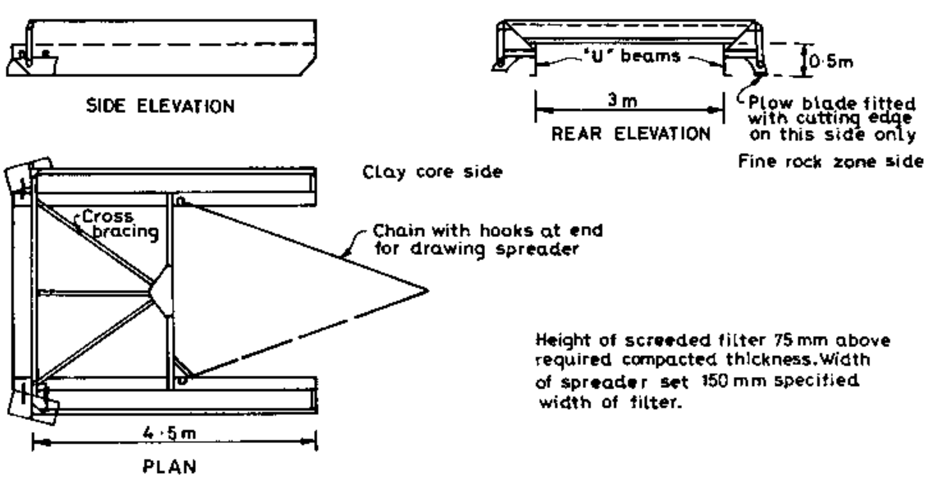


Figure 7.15. Typical spreader box and screed (adapted from Charlton & Crane 1980).

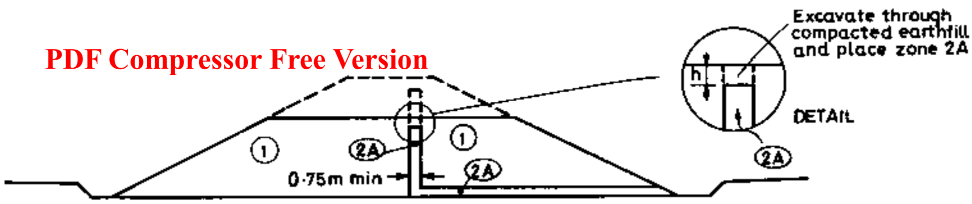


Figure 7.16. Construction of vertical chimney drain by excavation through earthfill.

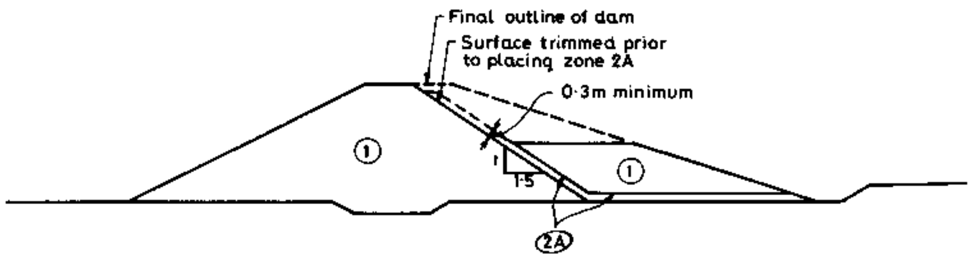


Figure 7.17. Construction of inclined chimney drain by placement on downstream slope of earthfill.

d) For homogeneous or zoned earthfill dams with a vertical chimney, a relatively narrow filter (as narrow as 0.75 or 1.0 m) can be constructed by placing the earthfill for up to 2 m over the filter layer, and then excavating through the earthfill with a backhoe or excavator to expose the filter, as shown in Figure 7.16. Careful cleanup of the surface of the exposed filter is necessary, and the filter is compacted with small vibrating sleds or other compaction equipment. The depth 'h' is best limited to say 1 m to reduce the risk of collapsing of the trench and allow access of men into the trench.

Contamination of the filter during placement can be reduced by spreading it from a movable steel plate placed adjacent the trench (Charlton & Crane 1980). This also reduces the risk of collapse of the trench under the surcharge load of the filter material.

e) For smaller dams, an inclined chimney drain can be constructed by dumping the filter on the trimmed downstream slope of the earthfill core as shown in Figure 7.17. The filter can be compacted by rolling up the slope or by running rubber tyred equipment up against the slope. The downstream earthfill (or rockfill) is then placed in layers adjacent to the filter. In this way thin layers of filter (say as thin as 0.3 m normal to the slope, 0.5 m horizontal) can be placed.

Horizontal filters can be placed in layers as thin as 150 mm or as thick as about 500 mm after compaction (the upper limit being determined by compaction requirements). However, for thick layers of filters (e.g. 400 mm), it is considered preferable to place 2×200 mm layers rather than 1×400 mm layer since, if segregation occurs during placement, it is less likely that coarse zones will coincide with each other in the two layers.

7.4.2 *Sequence of placement*

Filters generally should be placed ahead of the adjacent earthfill or rockfill zones as shown in Figure 7.18.

This is desirable because it allows good control of the width of the filter zone compared to

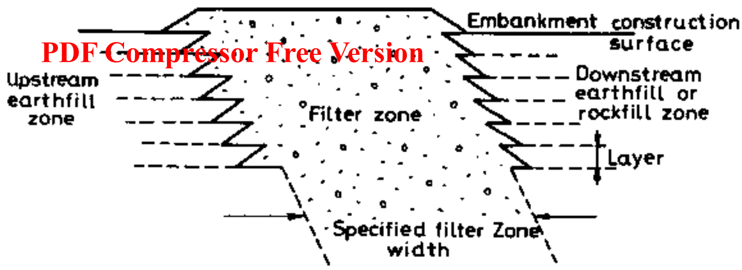


Figure 7.18. Filter zone placement ahead of other zones – generally desirable.

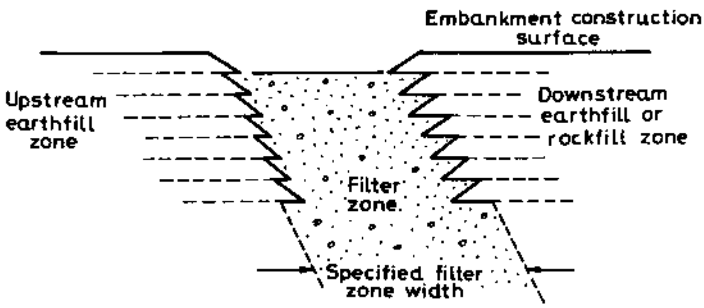


Figure 7.19. Filter zone following construction of other zones – generally undesirable.

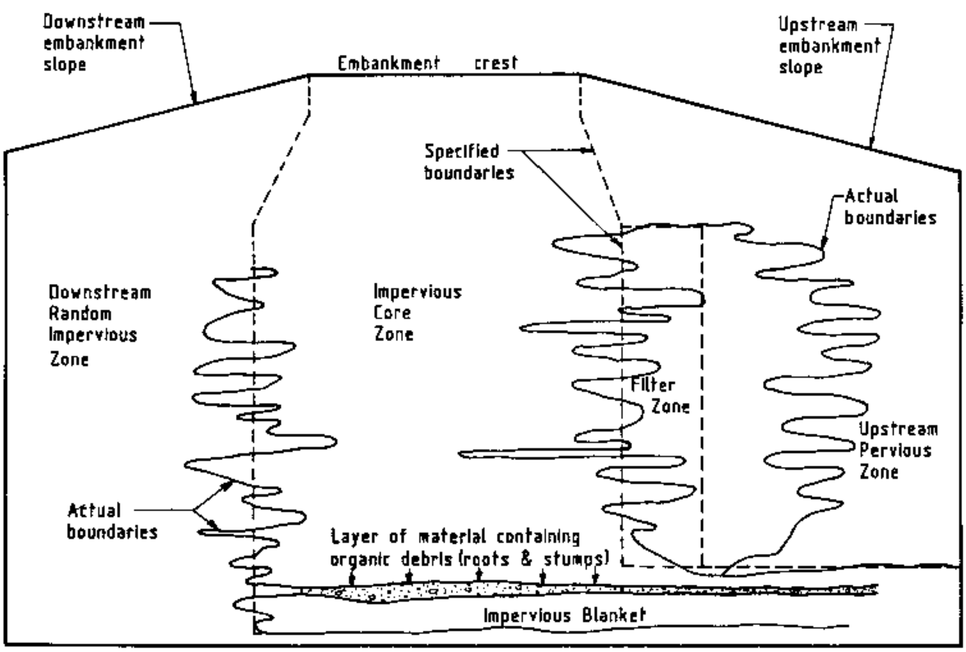


Figure 7.20. Embankment section of a dam with poor control of zoning boundaries during construction (Pritchett 1985).

the specified width, and reduces the risk of contamination of the filter zone with materials from the adjacent zones and from water eroding off adjacent areas generally. Figure 7.19 shows the alternative which is not recommended. However, both systems are used successfully by experienced contractors.

The need for careful survey and construction control of placement of filters was highlighted by the example given in Pritchett (1985) and reproduced in Figure 7.20. This 26 m high dam developed excessive seepage on first filling and the as-constructed zoning was determined by investigation excavation and mapping.

7.4.3 *Compaction of filters*

Filters should be compacted in layers using a vibratory smooth steel drum roller. Filters are usually well graded granular materials and are readily compacted to a dense condition. Hence for the majority of dams, a 'methods' type specification for compaction is the most practicable, i.e. a maximum layer thickness, coupled with a number of passes of a vibratory roller of a specified static weight and centrifugal force. Table 7.2 gives some examples 'standards' type specifications, where a density index (relative density) is specified. This is not preferred because of the cost involved in carrying out the tests, the delay in the results, and the difficulty of carrying out the testing required, particularly in Zone 2B filters. Zone 2B filters will commonly have gravel up to 50 or 75 mm size, requiring a density in place test hole at least 200 or 300 mm

Table 7.2. Typical filter compaction specifications.

Authority and dam	Zone	Standards specification	Methods specification
Water Resources Commission of Queensland, Peter Faust Dam (1989)	2A and 2B	AS1289. Density index between 60% and 70%	At least one pass of a suitably smooth drum vibratory roller to give standards specification. Maximum layer thickness 350 mm after compaction
Water Authority of Western Australia, Harris Dam (1989)	2A Chimney drain	AS1289. Density index > 70%	Not given. Maximum thickness 500 mm before compaction
	2A horizontal drain and 2B	Not specified	4 passes of a vibratory roller with static mass between 8 and 12 tonnes, and centrifugal force not less than 240 kN. Maximum thickness 600 mm before compaction
Melbourne and Metropolitan Board of Works, Cardinia Creek Dam (1970)	2A and 2B	Not specified	2 passes of a vibratory roller with static weight not less than 8 tonnes and a centrifugal force not less than 160 kN. Maximum layer thickness 450 mm after compaction
Snowy Mountains Hydroelectric Authority, Talbingo Dam (1967)	2A and 2B	Not specified	4 passes of a vibratory roller with static weight not less than 10 tonnes static weight and a centrifugal force not less than 350 kN. Maximum layer thickness 450 mm after compaction

diameter, and laboratory compactions in non standard 200 or 300 mm diameter cylinders.

An exception might be in severe earthquake zones where saturated filters might be subject to liquefaction unless compacted dense. In this case a requirement to compact to a density index of say 70 or 80% would be appropriate. It should be noted that requiring a density index greater than say 80% is likely to result in excessive breakdown of filters under the compactive effort and will achieve little or no benefit. There are some arguments, e.g. Sherard (1985), that over-compaction can lead to excessively high moduli for the filters, resulting in their settling less than the dam core during construction and first filling, and thereby reducing vertical stresses and horizontal stresses in the core. This can foster the development of hydraulic fracture. It is arguable whether it is practicable to set an upper limit on density index in dam construction such as for Peter Faust dam in Table 7.2, other than as a guide to contractors. It would seem difficult to insist on removal of over-compacted material without raising the issue of contractual claims.

7.5 USE OF GEOTEXTILES AS FILTERS IN DAMS

7.5.1 *Types and properties of geotextiles*

Koerner (1986) defines geotextiles as: 'Geotextile: a permeable textile material (usually synthetic) used with soil, rock or any other geotechnical engineering-related material to enhance the performance or cost of a human-made product, structure or system.'

A wide range of different geotextiles are available. These differ in the type of polymer used for manufacture, the type of fibre construction, and how this fibre is manufactured into a fabric. Koerner (1986) indicates that the following materials are used for manufacture of geotextiles (in decreasing order of use):

Polypropylene,
Polyester,
Polyamide (nylon),
Polyethylene,
Other polymers and glass.

The polymers are made into several different types of fibre:

Monofilament,
Multifilament (monofilament yarn),
Staple,
Staple yarn,
Slit film.

These fibres are then made into fabrics to produce non woven, woven or knitted fabrics.

Non woven fabrics are the most commonly used for filter applications and are manufactured by several different methods:

– Thermally bonded: Randomly laid fibres are held together by thermal fusion;

– Resin bonded: Randomly laid fibres are held together by glueing (and possible entanglement);

– Needle punched: Randomly laid fibres are held together by entanglement induced by needle punching.

Details of these processes are given in Koerner (1985, 1986).

When considering the use of geotextiles as filters in dam construction, the important properties are the 'particle size' as that affects its ability to act as a filter, and its 'permeability,' or ability to allow water to pass through. The 'particle size' of the geotextile is usually measured by sieving a standard soil or single sized glass spheres on the geotextile, for a fixed period, and observing the quantity and particle size of the soil which passes through. As described in Bertacchi & Cazzuffi (1985) and ICOLD (1986) there is no one accepted standard, and different methods use dry or wet sieving, and consider the geotextile size as that allowing zero, 2 or 5% of the soil passing.

Koerner (1986) suggests the use of the apparent opening size AOS (also known as equivalent opening size EOS), which is determined by a test procedure developed by the US Army Corps of Engineers. The test uses known size glass beads and determines by sieving using successively finer beads, that size of bead for which 5% or less pass the fabric. The AOS or EOS is the US standard sieve number of this size bead. It is also quoted as the equivalent sieve opening in millimetres or 95% opening size 0_{95}. This is the form used for most filter design criteria as discussed below. Koerner (1986) points out that the test has problems. Many manufacturers provide data on the 95% opening size in their product literature and it appears to have become the industry standard, at least for the time being.

Some typical values are:

Product name	Structure woven (W)	EOS (Micron)	Permeability cm/sec
Terrafirma s/2100	W	60-70	0.03
Polyfelt TS500	NW Needle P	212	0.5
Polyfelt TS800	NW Needle P	130	0.4
Bidim U14	NW Needle P	100-120	N/A
Bidim U64	NW Needle P	60-75	
Polytrac	W	70 sieve	

In most cases, the ability of the geotextile to transmit water across the fabric is also important. This is assessed as the permittivity

$$\psi = \frac{kn}{t}$$

where ψ = permittivity
kn = cross plane permeability coefficient
t = geotextile thickness at the normal pressure on the geotextile.
In Darcy's Law:

$$q = ki\,A$$
$$= kn\,\frac{\Delta h}{t} \cdot A$$

So

$$\frac{kn}{t} = \psi = \frac{q}{(\Delta h)\,A}$$

where q = flow rate
Δh = head loss across the geotextile
A = area of flow (i.e. of geotextile).

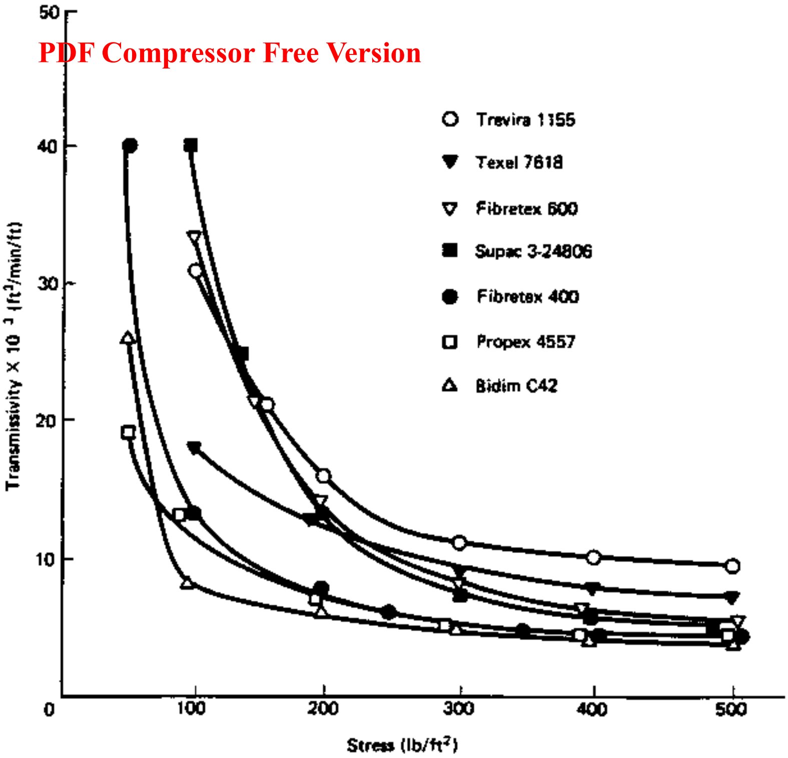


Figure 7.21. Transmissivity versus applied normal stress for various needle punched non woven geotextiles (Koerner 1986).

As pointed out by Bertacchi & Cazzuffi (1985) the permittivity is dependent on the normal stress applied to the geotextile i.e. the ability to transmit water is reduced as the geotextile is placed under stress which compresses the geotextile, and the values adopted for design should account for this. The permittivity is also affected by clogging of the geotextile by fine soil. This is discussed further below.

In some applications the geotextile may be used to transmit water along its plane i.e. it is in itself performing a drainage function. This is measured by transmissivity θ and is given by

$$\theta = kpt = \frac{qL}{\Delta h\ W}$$

where θ = transmissivity
kp = permeability in the plane of the fabric
t = thickness of the fabric
q = flowrate in the plane of the fabric
L = length of the fabric
W = width of fabric
Δh = head lost.

As shown in Figure 7.21, transmissivity is also affected by the applied normal stress.

Possible applications of geotextiles as filters in fill dams

Filter location	Purpose of filter	Type of flow or loading	Significance of failure	Access for repair
a. Downstream slope protection	Control of erosion by rainfall	Occasional surface flow	Non-critical	Easy
b. Downstream surface drains	Removal of surface seepage	Continuous local seepage	Non-critical. Local wet areas may reappear	Easy, possible
c. Upstream slope protection	Control of erosion by wave action and by outward flow during drawdown	Cyclic flow during wave action. Small flow during drawdown		
d. Temporary internal drainage	Dissipation of excess pore pressure during construction of wet fills	Temporary flow, limited quantity. Some migration of fines allowable if drains not blocked	Non-catatrophic. Failure may lead to instability during construction or delays	None
e. Upstream internal fill boundary on foundation contact	Prevention of unacceptable migration of fines in upstream direction	Transient and small flows during drawdown	Non-catatrophic. Only significant if migration is large and continuous	None
f. Downstream internal interface, no continuous flow from reservoir, e.g. beneath weight block	Prevention of unacceptable migration of fines	Flow only due to infiltration of rainfall	Limited and non-catastrophic	May be possible to excavate with reservoir drawn down for safety
g. Downstream internal interface, continuous flow from reservoir, e.g. downstream core boundary or foundation interface near core	Prevention of internal erosion, including effects of concentrated flow in cracks, etc.	Continuous flow from reservoir, potentially large and increasing	Potentially catatrophic and rapid. General seepage from downstream alone may involve only slow deterioration	Generally none. Downstream weight/block inverted filter may be removed and repaired with reservoir drawn down for safety

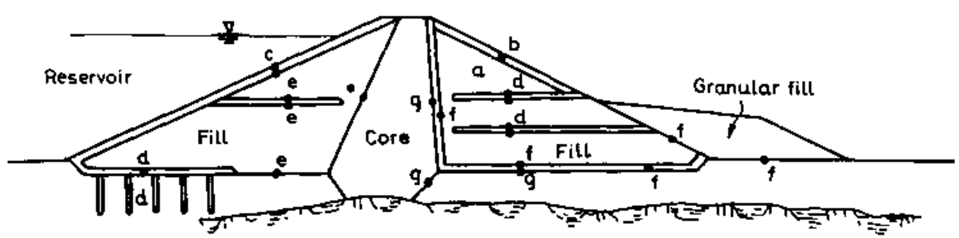


Figure 7.22. Filter functions as defined by ICOLD (1986).

7.5.2 *Filter design criteria*

There are many different filter design criteria for geotextiles. Summaries are given in Bertacchi & Cazzuffi (1985) and ICOLD (1986). Bertacchi & Cazzuffi (1985) indicate that they prefer the method proposed by CFGG (1983).

ICOLD (1986) conclude that: 'The following conclusions for filter design with geotextiles in dams can be tentatively drawn:

1) The hydraulic conditions and the critical nature of the interface concerned should be carefully considered and the function of the filter defined (see Fig. 7.22).

2) For non-cohesive base soils in one-way flow: (a) The geotextile should retain the D_{85} size of the base soil. This requirement is less conservative than typical rules for granular filters involving the D_{85} base soil particle size. (b) If the base soil is not well graded a finer geotextile opening size may be required to prevent excessive particle movement before a stable interface develops. (c) Particular care should be taken in adopting the design base soil grading for uniform soils, as a small error may result in all the particles of the base soil being smaller than the geotextile opening size. There is much less risk with base soils which are well graded in their coarse sizes. (d) Individual tests should always be performed where base soils are gap graded or have unusually shaped gradings and conservatism should be applied. Individual tests are also desirable for all important applications.

(3) For alternating or turbulent flow with non-cohesive base soils, much more conservative criteria are required.

(4) For cohesive base soils with non-dispersive conditions at the filter interface, and without concentrated continuous flow through cracks or similar defects, a fine pored geotextile is satisfactory.

(5) If continuous flow can occur through a crack or other opening in a cohesive soil, then the present state of the art is uncertain, a sand sized filter is generally more conservative to protect against internal erosion than a geotextile and it is doubtful if, currently, a geotextile should be relied upon.'

Unfortunately apart from 2 (a), no clear recommendations are made to relate the particle size of the soil to the 0_{95} for the geotextile.

The ICOLD (1986) recommendations appear to be significantly influenced by the work of the CFGG, which in turn are reported in Loudiere et al. (1982). Loudiere et al give more specific recommendations which are shown in Table 7.3 along with other methods.

Koerner (1986) suggests that the AASHTO (1983), Carroll (1983) or Giroud (1982) methods can be used, depending on the criticality of the situation, with Giroud giving the most conservative answers.

The ISSMFE Technical Committee on Geotextiles reported in Giroud et al. (1985) recommend use of the Heerten (1984) approach, or FHWA (1985).

Based on the work by Sherard and Dunnigan for sand filters, the authors have a preference for design methods which are based on the D_{85} of the soil, rather than D_{50} as required by Giroud. They are strongly influenced by the ICOLD committee recommendations. It is therefore recommended that for design of filters in dams the ICOLD recommendations be followed with assistance from the Loudiere method to quantify the requirements. A check of design using Heerten (1984) would seem appropriate, particularly for severe (dynamic) load conditions such as when geosynthetics are used as a filter under rip rap.

Table 7.3. Methods for designing geotextiles as filters.

Loudiere et al. (1982

For non cohesive soils:

uniformity coefficient $(D_{60}/D_{10}) > 4$

$O_{95} < D_{95}$

uniformity coefficient < 4

$O_{95} < 0.8\ D_{50}$

For cohesive soils:

uniformity coefficient > 4

$O_{95} < D_{85}$, and

$O_{95} \nless 50\ \mu m$

uniformity coefficient < 4

$O_{95} < 0.8\ D_{50}$, and

$O_{95} \nless 50\ \mu m$

AASHTO (from Koerner 1986)

For coarse grained soils, ≤ 50%, passing 75 μm

$O_{95} \leq 0.6$ mm

For fine grained soils, > 50%, passing 75 μm

$O_{95} \leq 0.3$ mm

(Note: Koerner quotes ≥ not ≤ for these relationships, which appears to be in error)

Carroll (1983)

$O_{95} < (2 \text{ or } 3)\ D_{85}$

Giroud (1982) (for needle punched and woven geotextiles)

Relative density	$1 < C_u < 3$	$C_u > 3$
Loose (RD < 35%)	$O_{95} < (C_u)\ D_{50}$	$O_{95} < 9\ D_{50}/C_u$
Medium (RD 35% to 65%)	$O_{95} < 1.5\ (C_u)\ D_{50}$	$O_{95} < 13.5\ D_{50}/C_u$
Dense (RD > 65%)	$O_{95} < 2\ (C_u)\ D_{50}$	$O_{95} < 18\ D_{50}/C_u$

For woven and heat bonded non wovens

	$1 < C_u < 3$	$O_{95} < (C_u)\ D_{50}$
	$C_u > 3$	$O_{95} < 9\ D_{50}/C_u$

where C_u = uniformity coefficient = D_{60}/D_{10}

Heerten (1984)

For cohesionless soils

static load conditions, $C_u \geq 5$

$O_{98} < 10\ D_{50}$, and

$O_{98} \leq D_{90}$

static load conditions, $C_u < 5$

$O_{98} < 2.5\ D_{50}$, and

$O_{98} \leq D_{90}$

dynamic load conditions (high turbulence, wave action or 'pumping')

$O_{98} < D_{50}$

For cohesive soils

$O_{98} < 10\ D_{50}$, and

$O_{98} \leq D_{90}$, and

$O_{98} \leq 0.1$ mm

7.5.3 *Permeability design criteria*

In keeping with the design approach for granular filter materials, it is necessary to consider the permeability of the geotextile as compared to the permeability of the filter.

In doing this, it must be recognised that experience with geotextiles has shown that they tend to become partially clogged with fine particles, and may also be clogged with iron compounds. ICOLD (1986) list many examples where geotextiles have become partially clogged. Heerten (1984) reports that permeability tests on samples of geotextile dug up from an in-service condition in seawalls gave lower values than short term clogging tests in the laboratory. Figure 7.23 shows the degree of clogging as measured by porosity (n and n′) and permeability (kn and kn′) for needle punched non woven textiles.

It can be seen that the permeability is reduced by 5 to 70 times but in these cases remained reasonably high at 10^{-3} to 10^{-4} m/sec which was still 5 to 10 times higher than the soil permeability. Woven geotextiles in the same application were more affected giving geotextile permeabilities 0.16 to 1.8 times the soil permeability.

Heerten (1984) suggests the use of a permeability reduction factor (given in ICOLD (1986) to estimate the clogged compressed permeability and suggests that the clogged geotextile permeability should be greater than the soil permeability.

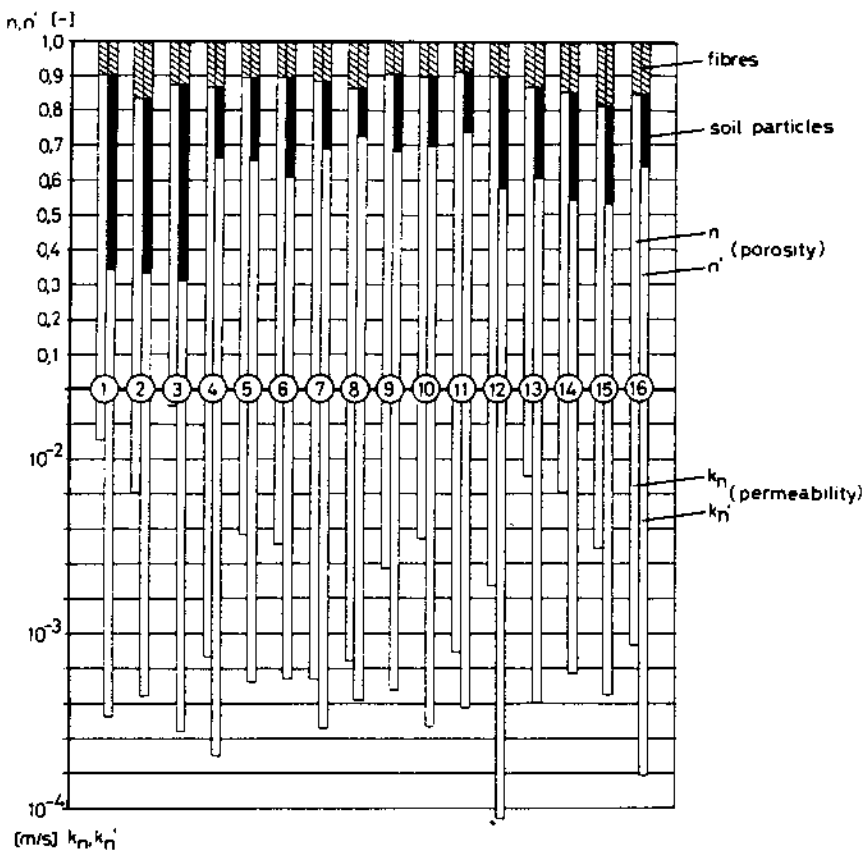


Figure 7.23. Clogging of voids measured for needle punched non woven geotextiles in service in seawalls (Heerten 1984).

Loudiere et al. (1982) reflecting the view of the CFGG recommend that the uncompressed new geotextile should have a permeability 100 ks where ks is soil permeability. This is described as 'very stringent' by ICOLD (1986). Bertacchi & Cazzuffi (1985) indicate that the CFGG (1983) proposed an equivalent permittivity for uncompressed new geotextile of 100 000 ks where ks is in m/sec and permittivity is in sec^{-1}.

In the absence of better information it is recommended that the CFGG requirements be met and that needle punched, non woven geotextiles be used because they are less likely to clog. Based on their own experience and that outlined below, the authors warn that mine tailings can cause major clogging of geotextiles, and they should not be used in direct contact with tailings.

7.5.4 *Use of geotextiles in embankment dams*

Geotextiles are often used as construction expedients – in roads, erosion control etc. Their use in dam construction in these situations is not contentious.

Use of geotextiles to perform a permanent function, probably in lieu of a sand/gravel filter, does require consideration of the effectiveness of the geotextile as a 'filter' and 'drain' as has been discussed in preceding sections. It also requires consideration of long term function, and durability, particularly if the geotextile is performing a critical function and is not accessible for checking its condition or for repair.

Figure 7.22 from ICOLD (1986) outlines possible uses of geotextiles in dam construction.

ICOLD (1986) conclude that:

– Considerable caution is required in using geotextiles to prevent erosion at interfaces which are subject to seepage from the reservoir (i.e. location 'g' in Fig. 7.22).

– Successful use in non critical locations, e.g. under slope protection (as in 'c' in Fig. 7.22) should not be used as justification for use in critical areas.

– Geotextiles are subject to damage during construction, particularly from angular coarse grained materials. This may have little affect on their effective use as a separator of different soils during construction but continuity is necessary for effectiveness as a filter.

– Geotextiles once buried have similar durability to other man made materials. However they are subject to rapid deterioration if left exposed to sunlight during or after construction.

A review of papers in international conferences on geotextiles shows several examples of the use of geotextiles in dam construction: Biche (1986) describes the use of a needle punched non woven textile in a 145 m high dam in Morocco. The geotextile 'Bidim' was used as a filter in the coffer dam, and as a separation layer between fine and coarse filters in the horizontal drain.

Deatherage et al. (1987) describe the use of a woven polypropylene geotextile (Mirafi 500X) on the upstream face of a vertical drain, and for a toe drain, and a geomembrane (Mirafi MCF 500) on the downstream face of the vertical drain. The vertical drain was installed as remedial measures after the (levee bank) 15 km long flood control dam was deliberately breached for safety reasons because of extensive cracking in the dam.

Dib & Agwar (1987) describe the use of a needle punched non woven geotextile (Bidim) in the cutoff trench for a dam in Brazil. As shown in Figure 7.25 the geotextile was used on the downstream slope of the cutoff trench in conjunction with slush grouting to control erosion of the earthfill in the cutoff trench into tubular holes in the soil resulting from weathering and leaching of metabasic rocks. The geotextile was required to span any undetected and unfilled (with grout) holes up to 50 mm dia.

List (1982) describes the use of geotextile as a filter drain downstream of a narrow soil cement core in two dams constructed in the Federal Republic of Germany. The dams were 33 and 86 m high. The geotextile is progressively placed on the downstream side of the core trench as the dam is raised.

Scheurenberg (1982) describes the use of geotextiles in underdrains in tailings dams in South Africa. Details are shown in Figure 7.27.

Type A was the original design which did not include geotextiles. Types B, C and D drains have proven successful for periods up to 5 years except where the geotextile with some 'slimes' on it had been exposed to the atmosphere. In these areas iron oxides precipitated, blocking the geotextile.

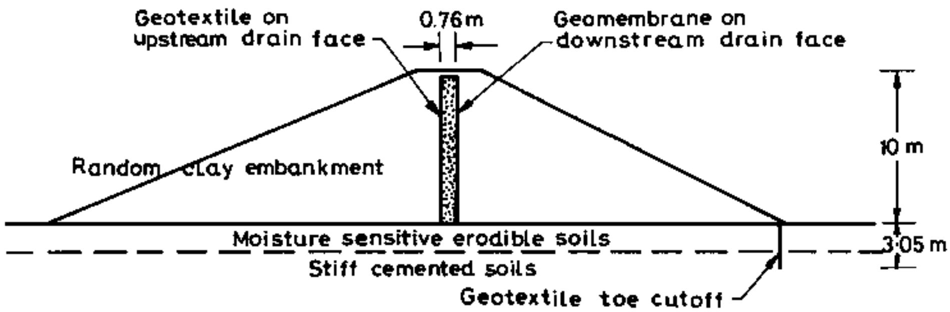


Figure 7.24. Use of geotextile for remedial works in a flood control dam (Deatherage et al. 1987).

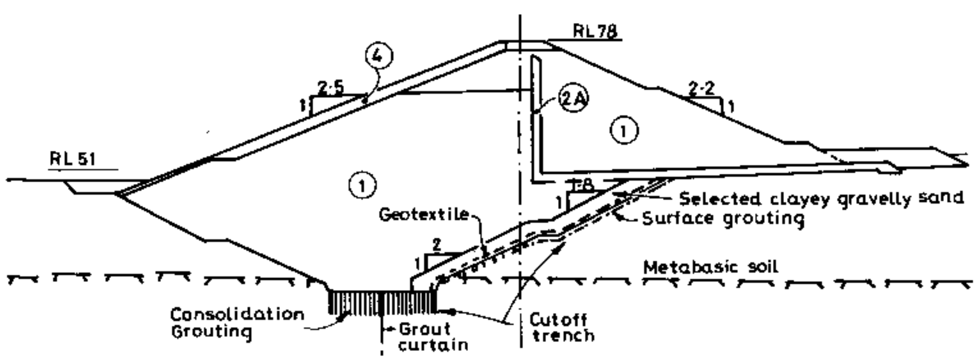


Figure 7.25. Use of geotextile in the cutoff trench for a dam in Brazil (Dib & Agwar 1987).

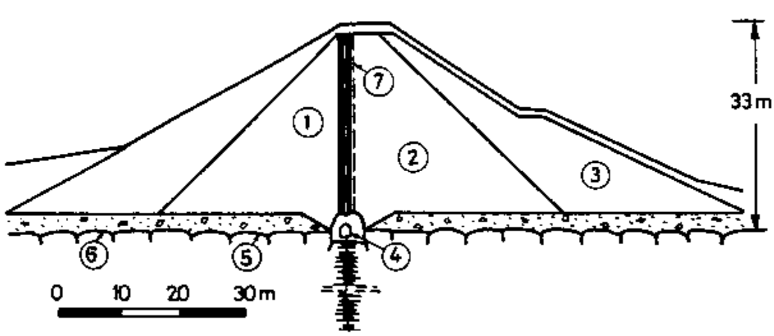


Figure 7.26. Use of geotextile downstream of soil-cement central core 1) Dry diaphragm wall, 2) Semi-impervious material, 3) Rockfill shoulders, 4) Grouting and control gallery, 5) Alluvium, 6) Bedrock, 7) Geotextile (List 1982).

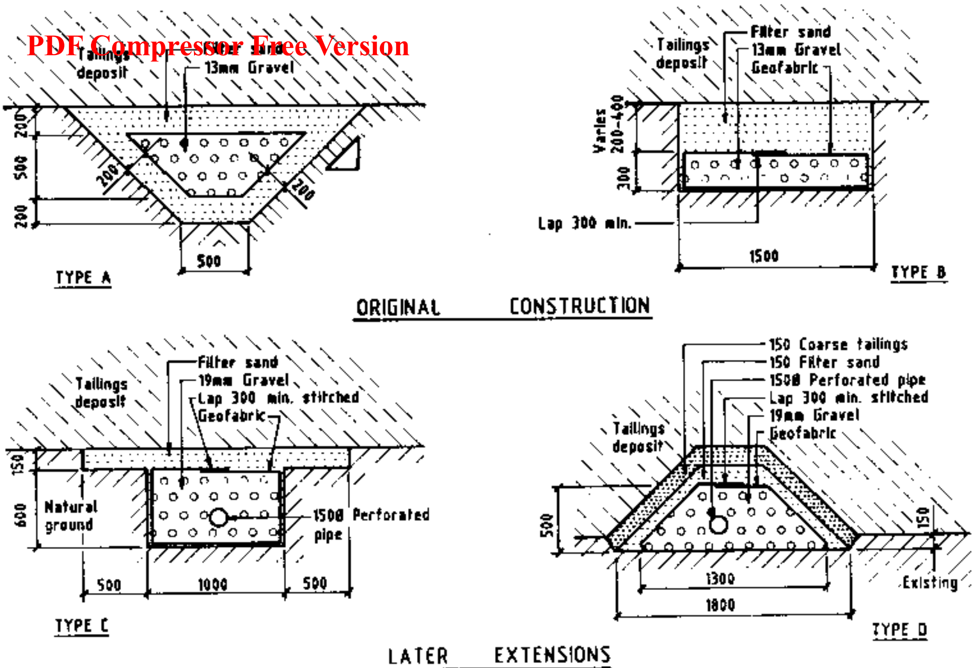


Figure 7.27. Use of geotextiles as underdrains in tailings dams in South Africa (Scheurenberg 1982).

Bentel et al. (1982) also describe their use of geotextiles as underdrains in tailings dams. They offer some useful practical hints based on their experience

– joints in the geotextile should be stitched, not just lapped;

– careful trimming and compaction of the foundation on which the geofabric is to be laid is recommended to give uniform support;

– pretreatment of the foundation with weed killer is recommended to prevent damage by weeds and other vegetation;

– filter sand should be placed over geofabric to prevent damage by sunlight in the period before being covered by tailings, and to prevent clogging of the geotextile by tailings or fine wind blown sand.

Figure 7.28 shows several drain designs used by Bentel et al. (1982). The relative costs for the drain in 1982 dollars were (a) \$45/m; (b) \$40/m; (c) \$30/m; (d) \$35/m.

Bertacchi & Cazzuffi (1985) also give some examples of use of geotextiles in dam construction, basically as part of a geomembrane system for the waterproof element in drains. ICOLD (1986) give an extensive list of uses of geotextiles in drain construction but few details are given.

The authors' opinion is that geotextiles do have a place in replacing sand and gravel filters in embankment dam construction, but only in non critical areas i.e.

– under rip-rap or other slope protection where damage can be repaired,

– in temporary internal drainage layers as shown in Figure 7.22.

Use in some critical areas such as above a horizontal drain ('f' in Fig. 7.22) may be acceptable in some cases. The authors would not use geotextiles in critical areas ('g' in Fig. 7.22) in other

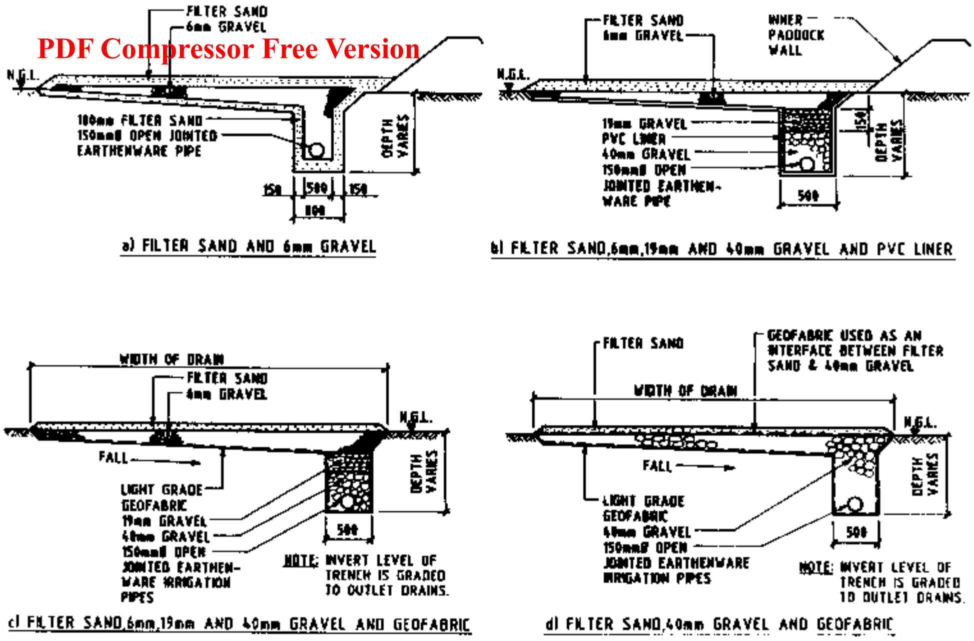


Figure 7.28. Use of filter sand and geotextiles for underdrains in tailings dams (Bentel et al. 1982).

than low hazard dams which are constructed to have a limited useful life – as sometimes is the case in mining operations. The use as filter underdrains in tailings dams is considered appropriate in most cases as after shutdown the drains do not need to be effective.

Geotextiles may also have application where a dam is expected to undergo differential movement, e.g. due to minor subsidence. Here the ability of the geotextile to 'bridge' cracks may be of some benefit as a backup to sand/gravel filters.

CHAPTER 8

Clay mineralogy, soil properties, dispersive soils and piping failure

8.1 INTRODUCTION

The properties of soils are determined by the properties of the constituent soil particles, the nature and quantity of water in the soil, the past consolidation history of the soil and soil structure. In this chapter the influence of the mineralogy of clays present in fine grained soils is discussed, with a particular emphasis on dispersive soils, and their use in the construction of embankment dams.

Dispersive soils are those which by the nature of their mineralogy, and the chemistry of the water in the soil, are susceptible to separation of the individual clay particles and subsequent erosion of these very small particles through fine fissures or cracks in the soil under seepage flows.

This is distinct from erodible soils, such as silt and sand, which erode by physical action of the water flowing through or over the soil.

For many years it has been recognised (e.g. Aitchison et al. 1963, Aitchison & Wood 1965) that the presence of dispersive soils either in the soil used to construct a dam, or in the dam foundation, greatly increases the risk of failure of the dam by 'piping failure,' i.e. development of erosion to the extent that a hole develops through the embankment, with rapid loss of water from the storage. Figure 8.1 shows such a case.

An understanding of the basic concepts of clay mineralogy is essential to the understanding of identification and treatment of dispersive soils in dam engineering. The following gives an outline of the subject. For more details the reader is directed to Mitchell (1976) and Grim (1963). Holtz & Kovacs (1981) give a useful summary of the topic.

8.2 CLAY MINERALS AND THEIR STRUCTURE

8.2.1 *Clay minerals*

The basic 'building blocks' of clay minerals are silica tetrahedra and aluminium (Al) or magnesium (Mg) octohedra. These give sheet like structures as shown in Figure 8.2. The alumina octohedra are known as gibbsite, the magnesium octohedra, brucite.

These in turn combine to give the clay minerals. Figure 8.3 shows the structure of montmorillonite and kaolin.

Some silicate clay minerals do not have a crystalline structure, even though they are fine

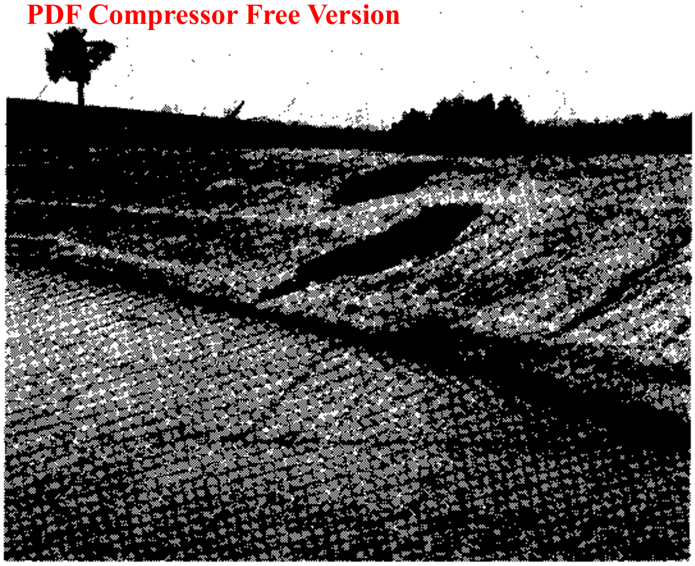


Figure 8.1. Examples of piping failure of a dam due to the presence of dispersive soils (Soil Conservation Service of NSW).

grained and display claylike engineering properties. These are known as allophane and are present in most soils. Mitchell (1976) indicates they are particularly common in some soils formed from volcanic ash.

Oxides also occur widely in soils and weathered rock as fine grained particles which exhibit claylike properties.

Examples are:

– gibbsite, boehmite, hematite and magnetite (oxides of Al, Fe and Si which occur as gels, precipitates or cementing agents)

– limonite: Amorphous iron hydroxide

– bauxite: Amorphous aluminium hydroxide.

8.2.2 *Bonding of clay minerals*

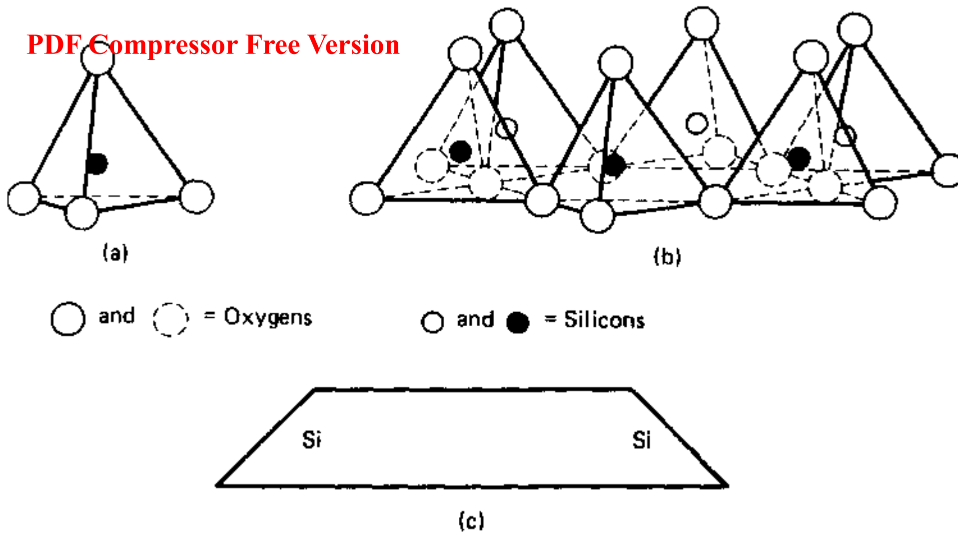
The properties of clays, particularly in a soil-water environment, are determined by the nature of the atomic bonding between the atoms make up the sheets of clay mineral, and the bonding between the sheets of clay mineral. The bonding mechanisms which are described in the following two sections.

8.2.2.1 *Primary bonds*

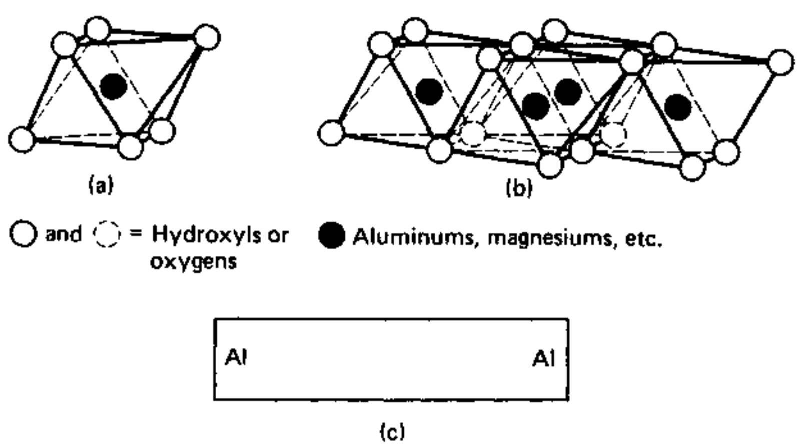
These are either:

(i)Covalent bonds, where two atomic nuclei share one or more electrons, e.g. H• + H• = H:H, or the hydrogen molecule

(ii)Ionic bonds, which result from the electrostatic attraction between positive and negative



(a) Single silica tetrahedron, (b) Isometric view of the tetrahedral or silica sheet, (c) Schematic representation of the silica sheet.



(a) Single aluminium (or magnesium) octahedron, (b) Isometric view of the octahedral sheet, (c) Schematic representation of the octahedral or alumina (or magnesia) sheet.

Figure 8.2. Silica tetrahedra, and aluminium and magnesium octohedra (Holtz & Kovacs 1981).

ions (ions are free atoms which have gained or lost an electron) e.g.

$$Na^{+} + Cl^{-} = NaCl$$

Positive charged ions are known as cations, negatively charged as anions.

When an ionic bond forms, the centres of negative and positive charge are separated and form a dipole. The dipole then has a positive and negatively charged 'end,' even though it is neutral overall.

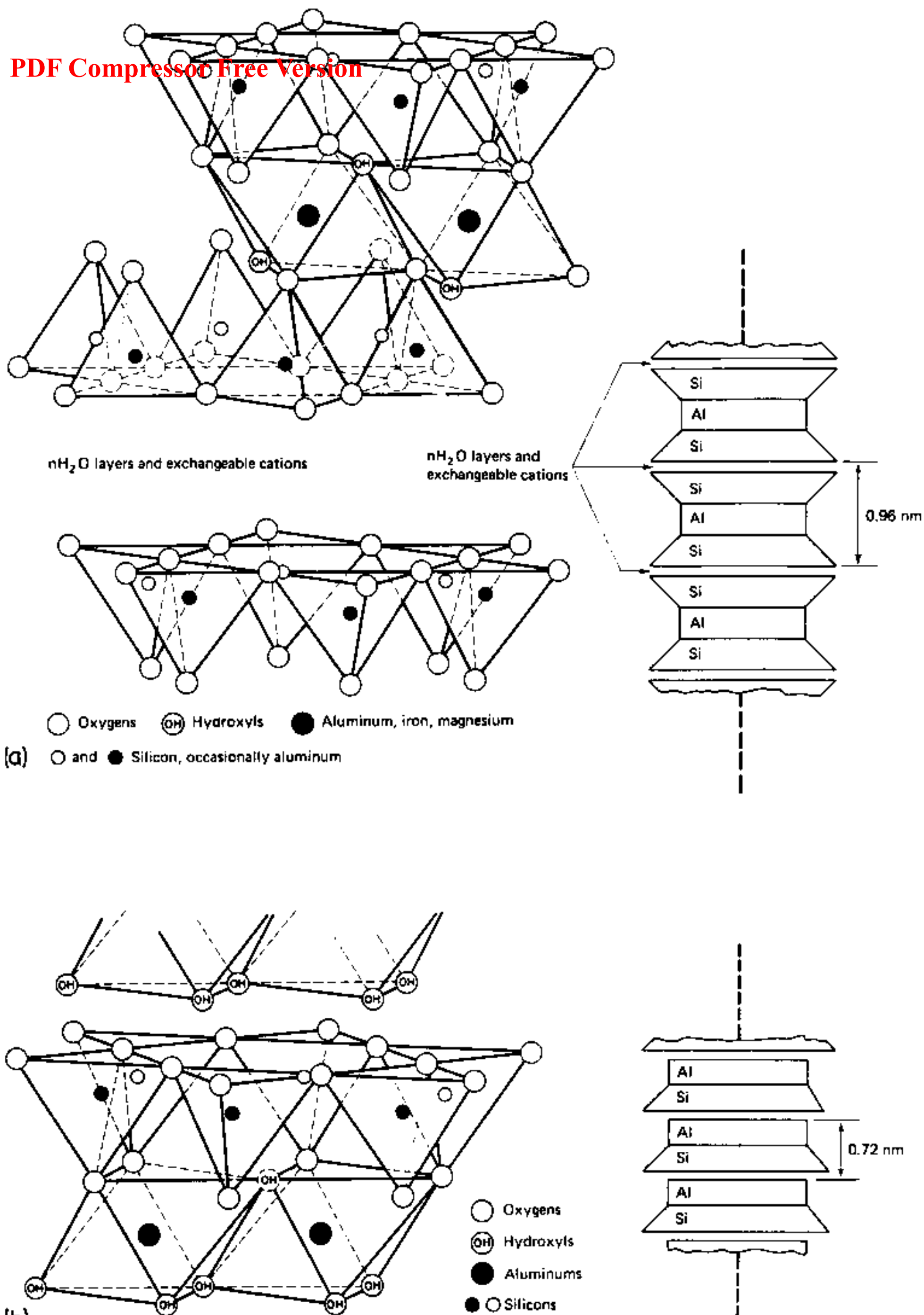


Figure 8.3. Structure of montmorillonite and kaolinite (Holtz & Kovacs 1981). a) montmorillonite; b) kaolinite.

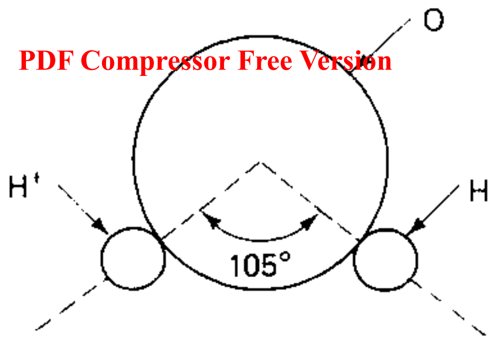


Figure 8.4. Schematic diagram of a water molecule showing its dipole nature (Holtz & Kovacs 1981).

Water is an example of a molecule which forms a dipole as shown in Figure 8.4. This dipolar nature has an important effect on the behaviour of water in clays.

Mitchell (1976) indicates that purely ionic or covalent bonds are not common in soils, and a combination is more likely. Silica ($Si0_2$) is partly ionic, partly covalent.

(iii)Metallic bonds, which are of little importance in the study of soils.

8.2.2.2 *Secondary bonds*

(i)Hydrogen bonds. Attraction will occur between the oppositely charged ends of permanent dipoles. When hydrogen is the positive end of the dipole, the resulting bond is known as hydrogen bonding.

(ii)Van der Waal's bonds. As the electrons rotate around the nucleus of an atom there will be times when there are more electrons on one side of the atom than the other, giving rise to a weak instantaneous dipole. This will not be permanent in one direction. The attraction of the positive and negatively charged ends of the fluctuating dipole causes a weak bond known as Van der Waal's bond. These bonds are additive between atoms, and although weak compared to hydrogen bonds, they decrease less with distance from the nucleus than the latter.

8.2.3 *Bonding between layers of clay minerals*

Bonding between layers of clay minerals occurs in five ways:

– Van der Waal's forces: These occur in most clay minerals.

– Hydrogen bonding, e.g. between the positive end of the OH at the base of the Al octohedra and the negative end of the O at the top of the silica tetrahedra in kaolinite (Fig. 8.3b).

– Hydrogen bonding with polar water, as shown in Figure 8.6.

– Exchangeable cation bonding: Cations, e.g. Ca^{++}, Na^{+} act to bond between the negatively charged surfaces of such clay minerals as montmorillonite. The negative charge is brought about by substitution of cations within the sheet structure e.g. Mg^{++} for Al^{+++}. This is known as isomorphous substitution and is permanent. The interlayer cations are not permanent and may be substituted by other cations.

– Interlayer cation bonding: Cations such as K^{+}, Mg^{++}, Fe^{++} which fit in the space between layers giving a strong bond. Examples are micas and chlorites. These cations are not affected by the presence of water. Figure 8.5 shows the structure of illite and chlorite.

Table 8.1 lists the predominant bonding for the common clay minerals.

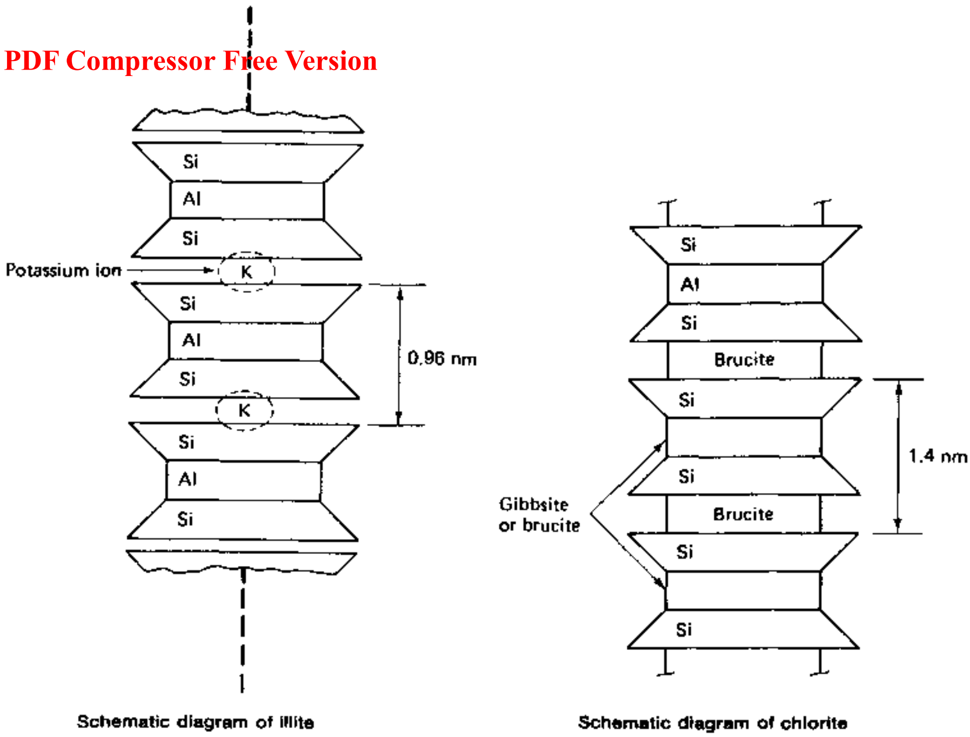


Figure 8.5. Schematic diagram of the structure of illite and chlorite molecule (Holtz & Kovacs 1981).

8.3 INTERACTION BETWEEN WATER AND CLAY MINERALS

8.3.1 *Adsorbed water*

There is much evidence that water is attracted to clay minerals, e.g:

– dry soils take up water from the atmosphere, even at low relative humidity;

– soils wet up and swell when given access to water;

– temperatures above 100°C are required to drive off all water from soils.

Much of the water in soil is adsorbed ie. the electrical charge of the water dipoles and the electrical charge associated with the surface of the clay minerals attract each other.

There are several explanations for this as shown in Figure 8.6:

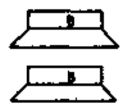
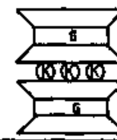
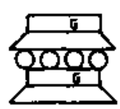
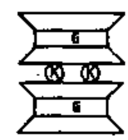
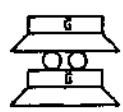
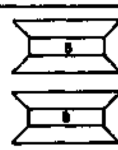
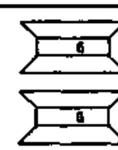
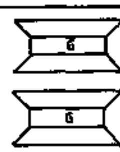
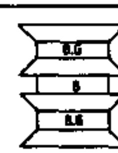
– hydrogen bonding, where the positive H side of the water dipole is attracted to surface oxygens, or the negative O side of the water dipole is attracted to surface hydroxyls;

– ion hydration, where water dipoles are attracted with cations, which are in turn attracted by the negative charge of the surface of the clay mineral;

– osmosis: As discussed below, there is a greater concentration of cations close to the surface of the clay than further away. Water molecules tend to diffuse towards the surface in an attempt to equalise concentrations;

– dipole attraction: Water dipoles orienting themselves as shown in Figure 8.6d, with a central cation to equalise charges.

Table 8.1. The structure of common clay minerals (adapted from Lambe & Whitman 1981).

MINERAL	STRUCTURE SYMBOL	MINERAL	STRUCTURE SYMBOL
SERPENTINE		MUSCOVITE	
KAOLINITE		VERMICULITE	
HALLOYSITE ($4H_2O$)		ILLITE	
HALLOYSITE ($2H_2O$)		MONTMORILLONITE	
TALC		NONTRONITE	
PYROPHYLLITE		CHLORITE	

Some clay minerals, particularly those of the smectite, or montmorillonite group, have undergone significant isomorphous substitution giving a large nett negative charge on the surface. They also tend to have particles which are very thin giving a large exposed surface area. As a result, these clays tend to attract water more than those which do not have a large negative surface charge and are thicker.

The surface area of clay minerals is measured by the specific surface, where

$$\text{specific surface} = \frac{\text{surface area}}{\text{volume}}$$

or

$$= \frac{\text{surface area}}{\text{mass}}$$

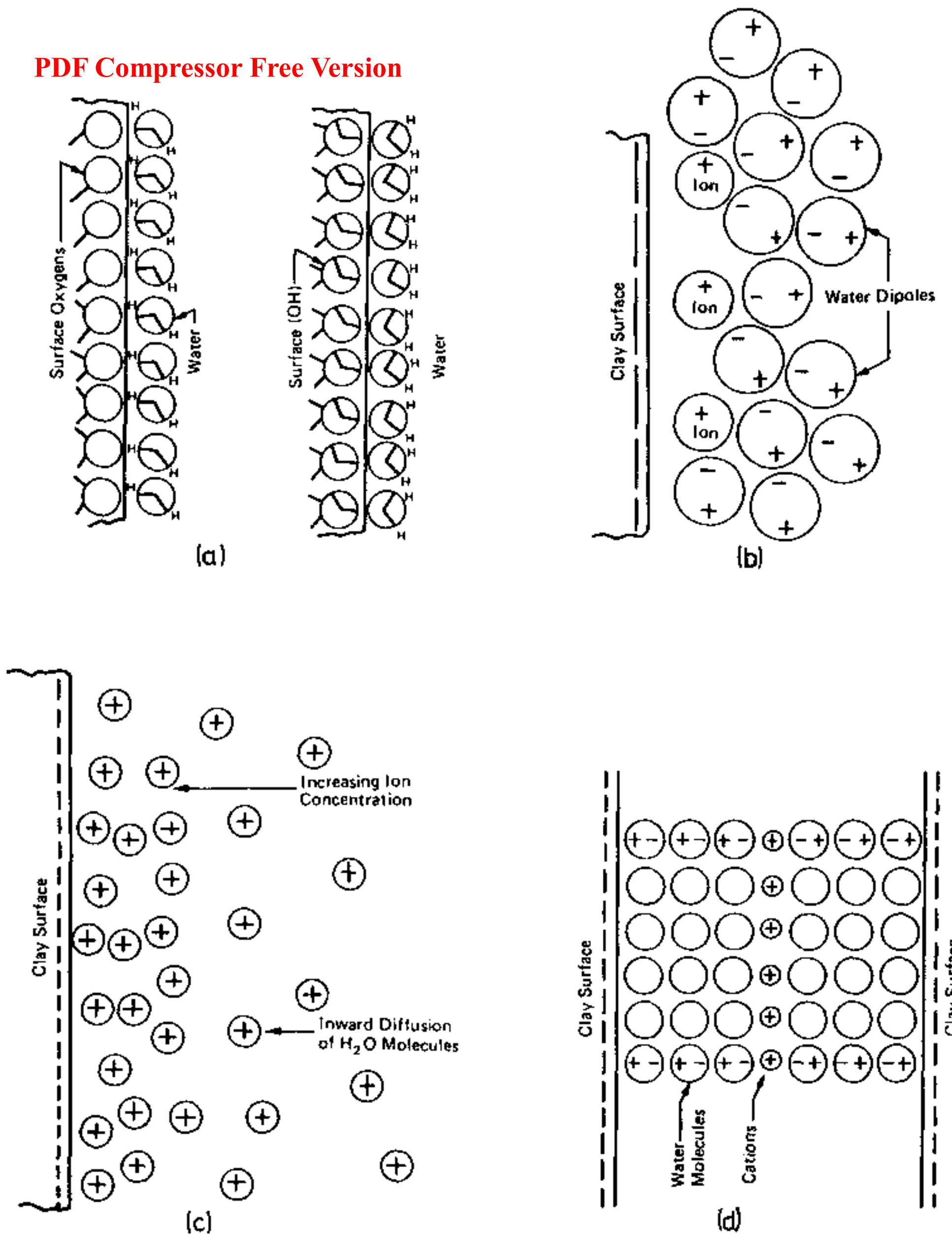


Figure 8.6. Possible mechanism of water adsorption by clay surfaces: a) hydrogen bonding; b) ion hydration; c) attraction by osmosis; d) dipole attraction (Mitchell 1976). Reprinted with permission of John Wiley & Sons Inc.

Edge View	Typical Thickness (nm)	Typical Diameter (nm)	Specific Surface (km²/kg)
Montmorillonite	3	100-1000	0.8
Illite	30	10 000	0.08
Chlorite	30	10 000	0.08
Kaolinite	50-2000	300-4000	0.015

Figure 8.7. Average values of the relative sizes, thicknesses and specific surfaces of the common clay minerals (Holtz & Kovacs 1981).

Figure 8.7 shows the typical dimensions and specific surface for four common clay minerals.

8.3.2 *Cation exchange*

As outlined above, one of the bonding mechanisms present in the montmorillonite (or smectite) group of clay minerals is exchangeable cation bonding. The cations, which are held in the space between sheets of clay minerals, are able to be exchanged for other cations, and, in the process, the properties of the clay can be altered. This process is known as cation exchange.

This source of cation exchange is the most important one in clay minerals except for kaolinite. Other sources are:

– Broken bonds: Electrostatic charges on the edges of particles and particle surfaces. This is the major cause of exchange in kaolinite.

– Replacement of the hydrogen of an exposed hydroxyl.

The most commonly present cations are Ca^{++}, Mg^{++}, Na^{+} and K^{+}. The ease with which the cations replace each other is dependent on:

– valence: Divalent ($^{++}$) substitute readily for univalent ($^{+}$), e.g. Ca^{++} for Na^{+};

– relative abundance or concentration of the ion type;

– ion size, or charge density: Smaller high charge cations will displace large, low charge cations.

A typical replacement series is

$$Na^{+} < Li^{+} < K^{+} < Mg^{++} < Ca^{++} < Al^{+++} < Fe^{+++}$$

However, Ca^{++} can be replaced by Na^{+} if the concentration of Na^{+} is sufficiently high.

The rate at which cation exchange occurs depends on:

– Clay type, for example, exchange will be almost instantaneous in kaolinite, a few hours in illite, and longer in montmorillonite.

– Concentration of cations in the soil water.

– Temperature.

The quantity of exchangeable cations required to balance the charge deficiency of a clay is known as the cation exchange capacity (CEC) and is usually expressed as milliequivalents per 100 grams of dry clay.

8.3.3 *Formation of diffuse double layer*

In a dry soil, the exchangeable cations are held tightly to the negatively charged clay surface. Excess cations and their associated anions precipitate as salts.

When placed in water, the salts go into solution and because of the high concentration of adsorbed cations near the clay surface these diffuse from the clay surface as they repel each other. However, this is counteracted by the negative charge on the clay surface. The anions in solution are repelled by the negative charge surface of the clay (but attracted to the cations). The overall effect is to result in a distribution of ions shown in Figure 8.8.

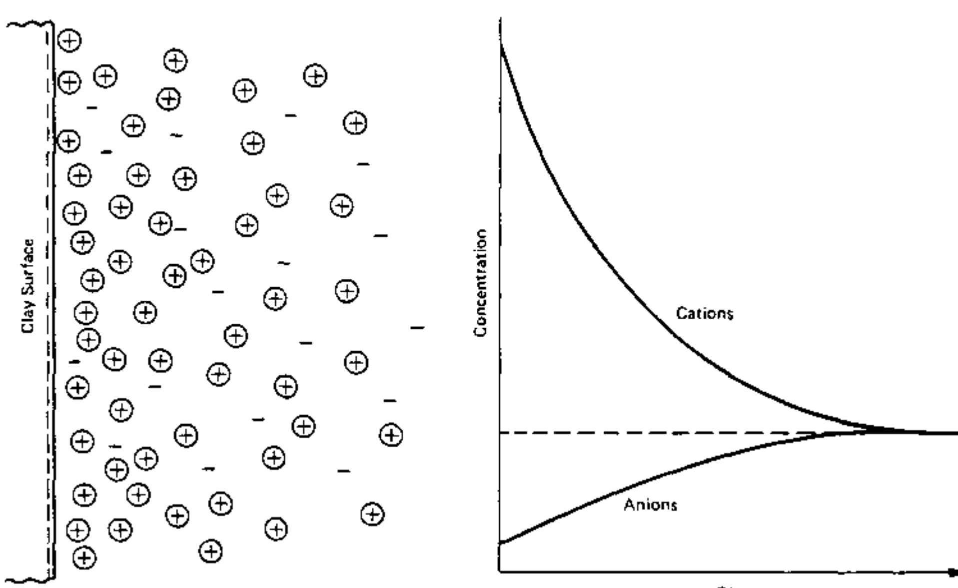


Figure 8.8. Distribution of ions adjacent to a clay surface according to the concept of the diffuse double layer (Mitchell 1976). Reprinted with permission of John Wiley & Sons Inc.

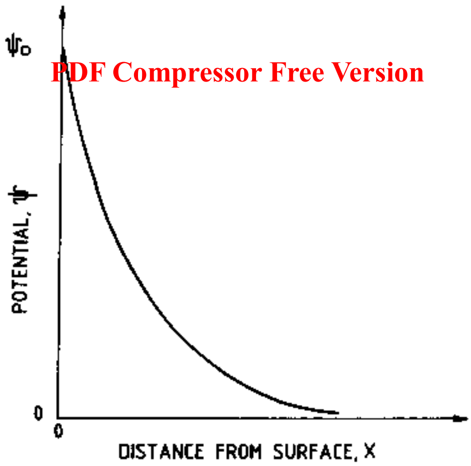


Figure 8.9. Variation in electrical potential with distance from the clay surface (Mitchell 1976). Reprinted with permission of John Wiley & Sons Inc.

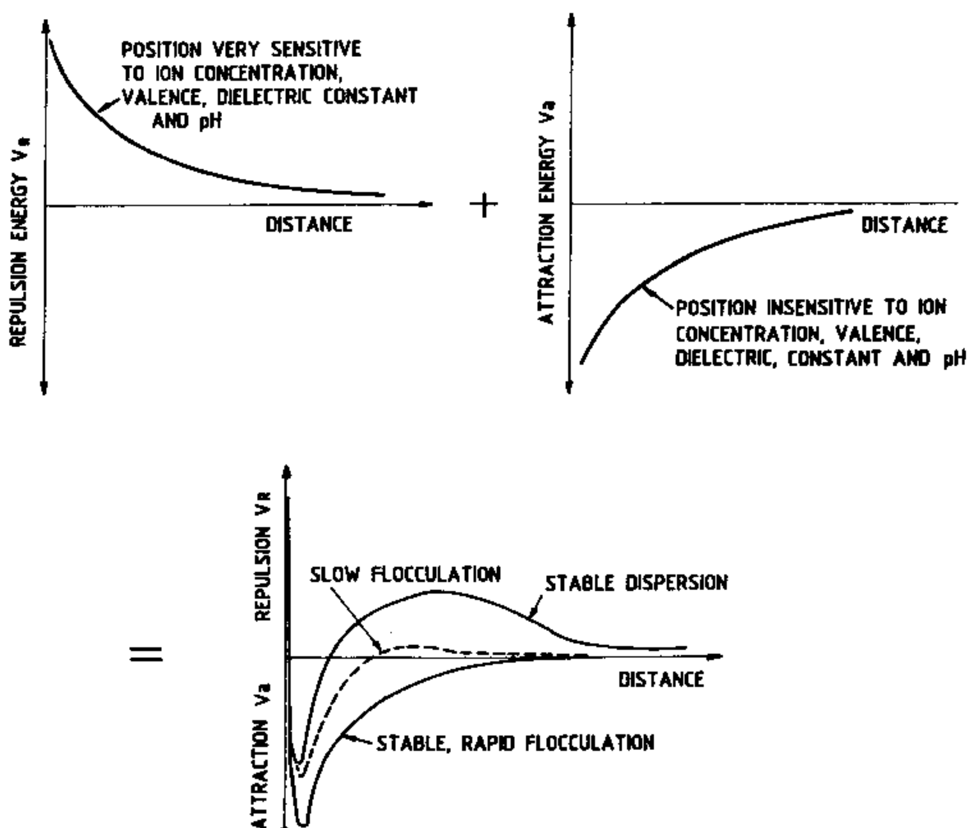


Figure 8.10. Interaction of repulsive and attractive forces (Mitchell 1976). Reprinted with permission of John Wiley & Sons Inc.

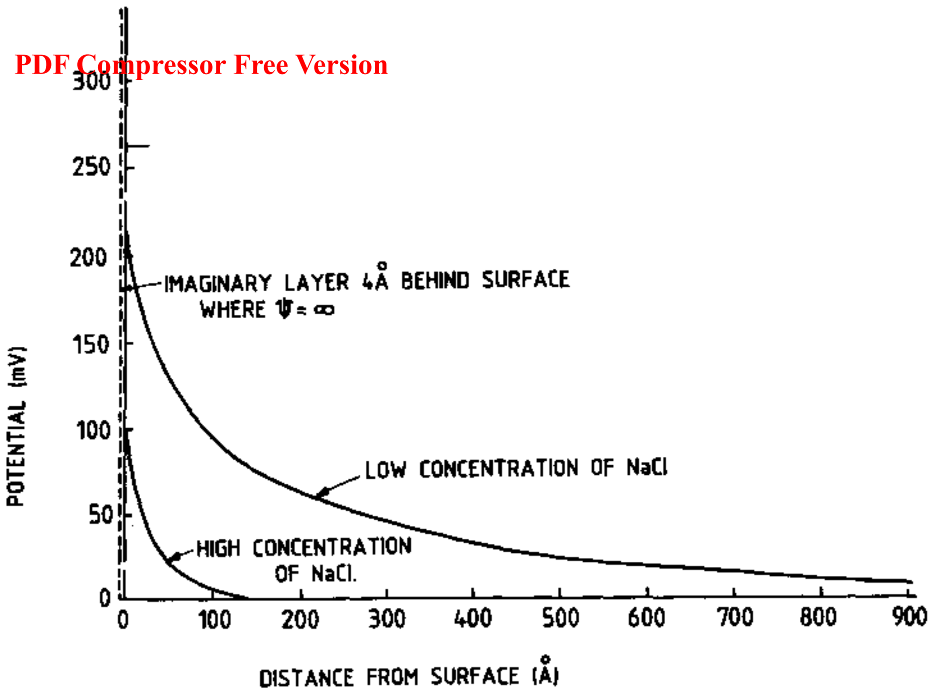


Figure 8.11. Effect of electrolyte concentration on diffuse double layer potential for montmorillonite (Mitchell 1976). Reprinted with permission of John Wiley & Sons Inc.

The negative surface and the distributed ions adjacent are together known as the diffuse double layer. Details are discussed in Mitchell (1976).

The resulting electrical potential varies with distance from the clay surface as shown in Figure 8.9. The potential is negative, reflecting the large negative charge on the clay surface.

8.3.4 *Mechanism of dispersion*

When two clay particles come near each other, the potential fields overlap, leading to repulsion if the particles are close enough. These repulsive forces are counteracted by Van der Waal's attractive forces as shown in Figure 8.10. If the repulsive forces are greater than the Van der Waal's forces the soil will disperse.

The repulsive forces in the diffuse double layer are affected by several factors:

(a) Electrolyte concentration. As shown in Figure 8.11, a high concentration of dissolved salt in the soil water leads to a smaller diffuse double layer (as the greater concentration of cations (Na^+) more readily overcomes the negative charge on the clay surface). Hence the repulsive forces are lower.

(b) Cation valence. As shown in Figure 8.12, exchange of Na^+ cations by Ca^{++} cations leads to a smaller diffuse double layer and hence lower repulsive forces.

Other factors which affect the diffuse double layer include:

– Dielectric constant of the electrolyte.

– Temperature.

More details are given in Mitchell (1976).

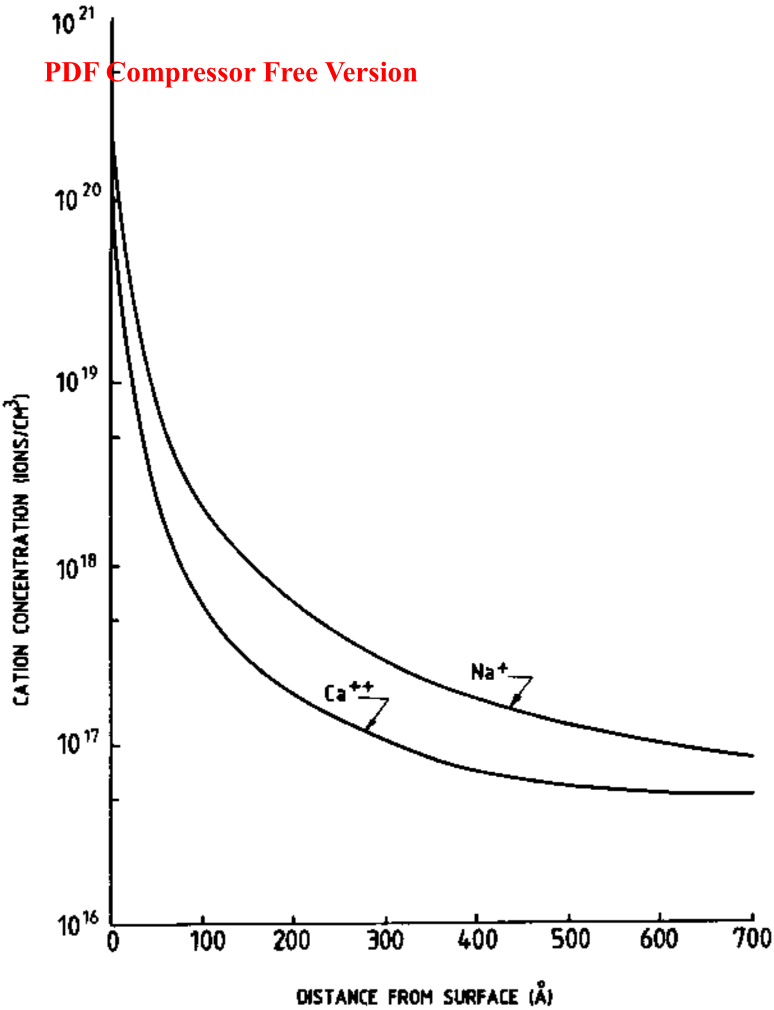


Figure 8.12. Effect of cation valence on diffuse double layer concentration for montmorillonite (Mitchell 1976). Reprinted with permission of John Wiley & Sons Inc.

8.4 IDENTIFICATION OF CLAY MINERALS

When required, the identification of clay mineral(s) present in a soil is usually carried out using at least two of the following techniques.

8.4.1 *X-ray diffraction*

A sample of the powdered soil is subject to X-rays, which are diffracted depending on the mineral crystals present. Each clay mineral has a characteristic pattern which is known. Figure 8.13 shows a typical X-ray trace.

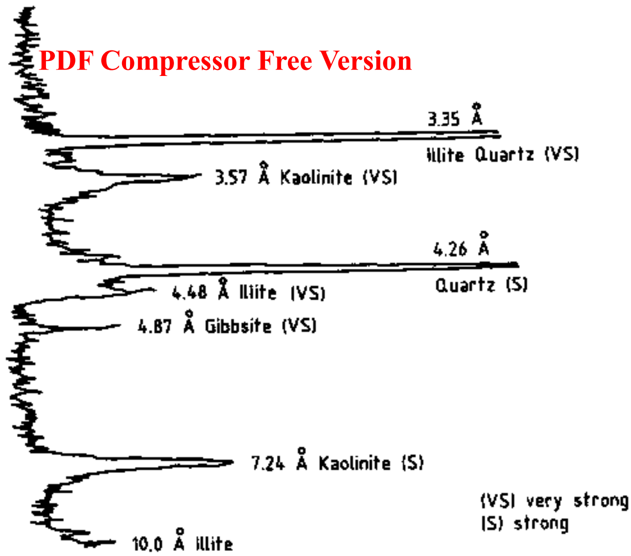


Figure 8.13. Typical x-ray diffraction trace (Lee et al. 1983).

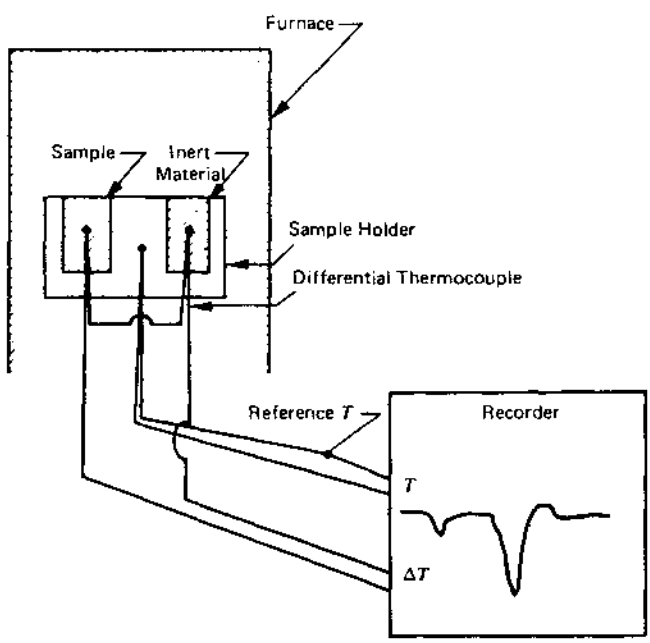


Figure 8.14. Schematic diagram of apparatus for differential thermal analysis (Mitchell 1976). Reprinted with permission of John Wiley & Sons Inc.

8.4.2 *Differential thermal analysis (DTA)*

A sample of the soil, and an inert substance are heated at a controlled rate (10°C/min) up to 1000°C, as shown in Figure 8.14.

The differences in temperature in the soil and the inert substance are recorded as shown in Figure 8.15.

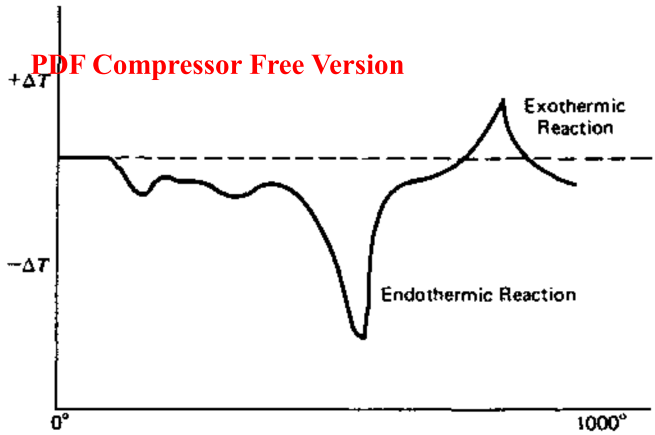


Figure 8.15. DTA thermogram for a sandy clay soil (Mitchell 1976). Reprinted with permission of John Wiley & Sons Inc.

Endothermic reactions (i.e. heat being absorbed) are usually related to the driving off of:
- adsorbed water,
- water of hydration

Exothermic reactions are usually related to recrystallisation and oxidation. Each clay mineral has a characteristic thermogram allowing its identification.

8.4.3 *Electron microscopy*

Scanning electron microscopy can be used to assist in identification of clay minerals. Magnifications from × 20 to × 150 000 can be used and the micrographs compared with those for known clay minerals. The main clay minerals have distinctly different micrographs.

In practice the presence of several clay minerals, and silt and sand complicates identification but, coupled with X-ray diffraction or differential thermal analysis, the method can be useful.

The above methods tend to be qualitative rather than quantitative. Mitchell (1976) indicates that an approximate quantitative analysis can be made using DTA, glycol adsorption, cation exchange capacity and K_20 content. The authors' experience is that few organisations are willing to quantify the amount of different clay minerals present.

Apart from the above methods a rough idea of which clay minerals are present can be obtained, see Section 8.4.4.

8.4.4 *Atterberg limits*

As shown in Figure 8.16 the position of the soil on the Casagrande plasticity chart can give an indication of which minerals are present.

As shown in Table 8.2, the Atterberg limits themselves can be used as a guide to the clay mineral present.

8.4.5 *The activity of the soil*

$$\text{Activity} = \frac{\text{plasticity index}}{\text{clay fraction}}$$

where clay fraction = % finer than 0.002 mm. Table 8.3 shows typical values.

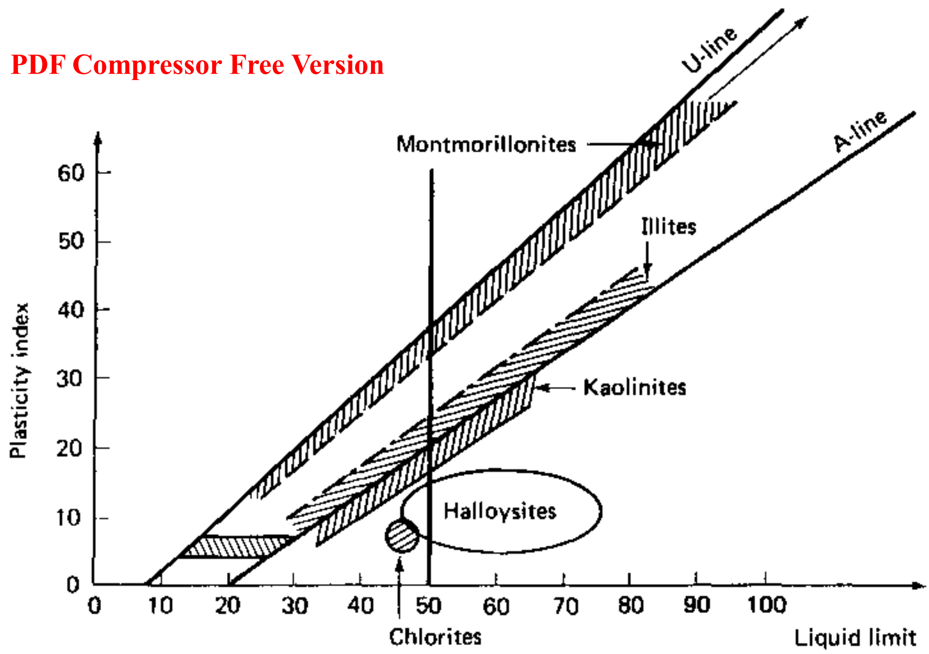


Figure 8.16. Location of common clay minerals on Casagrande plasticity chart (Holtz & Kovacs 1981).

Table 8.2. Atterberg limit values for the clay minerals (Mitchell 1976).

Mineral*	Liquid limit (%)	Plastic limit (%)	Shrinkage limit (%)
Montmorillonite (1)	100-900	50-100	8.5-15
Nontronite (1) (2)	37-72	19-27	
Illite (3)	60-120	35-60	15-17
Kaolinite (3)	30-110	25-40	25-29
Hydrated Halloysite (1)	50-70	47-60	
Dehydrated Halloysite (3)	35-55	30-45	
Attapulgite (4)	160-230	100-120	
Chlorite (5)	44-47	36-40	
Allophane (undried)	200-250	130-140	

*(1) Various ionic forms. Highest values are for monovalent; lowest are for di- and trivalent. (2) All samples 10% clay, 90% sand and silt. (3) Various ionic forms. Highest values are for di- and trivalent; lowest are for monovalent. (4) Various ionic forms. (5) Some chlorites are nonplastic.

8.5 ENGINEERING PROPERTIES OF CLAY SOILS RELATED TO THE TYPES OF CLAY MINERALS PRESENT

The engineering properties of clay soils depend on compositional factors:
- type of clay minerals present,
- amount of each mineral,
- type of adsorbed cations (and anions),

Table 8.3. Activities of various minerals (Holtz & Kovacs 1981).

Mineral	Activity
Na-montmorillonite	4-7
Ca-montmorillonite	1.5
Illite	0.5-1.3
Kaolinite	0.3-0.5
Halloysite (dehydrated)	0.5
Halloysite (hydrated)	0.1
Attapulgite	0.5-1.2
Allophane	0.5-1.2
Mica (muscovite)	0.2
Calcite	0.2
Quartz	0

– organic content,
– shape and size distribution of particles,
– pore water composition;

and on environmental factors:
– water content,
– density,
– confining pressure,
– fabric,
– availability of water,
– temperature.

Virtually all clay soils in nature are mixtures of clay and silt size particles (and sometimes sand), not just clay size particles. The silt and sand size particles are usually rounded or sub-rounded and are derived from the parent rock. The most abundant mineral present is usually quartz, followed by feldspar and mica.

8.5.1 *Dispersivity*

The dispersivity of a soil is directly related to its clay mineralogy. In particular soils with montmorillonite present tend to be dispersive, while kaolinite and related minerals (halloysite) are non dispersive. Illite tends to be moderately dispersive.

The dispersivity depends also on the pore water chemistry since, as discussed above, this affects the diffuse double layer geometry and electrical charge. In particular low electrolyte (pore water) salt concentrations lead to a large diffuse double layer and greater dispersivity. Hence percolation of a saline soil with fresh water can lead to dispersion.

Cation exchange of say Ca^{++} for Na^{+} leads to a smaller double layer, and lower dispersivity. Hence addition of lime (CaO or $Ca(OH)_2$, or gypsum ($CaSO_4$) leads to cation exchange and reduced dispersivity.

8.5.2 *Shrink and swell characteristics*

The shrink and swell characteristics of a soil can be related to clay mineralogy. Table 8.4 shows swelling index [de/d(logp)] where e = voids ratio, p = stress on soil) values for several clay

Table 8.4. Swelling index values for several minerals (Mitchell 1976).

Mineral	Pore fluid, adsorbed cations electrolyte concentration, in gram equivalent weights per litre	Void ratio at effective consolidation pressure of 100 psf	Swelling index
Kaolinite	Water, sodium, 1	0.95	0.08
	Water, sodium, 1×10^{-4}	1.05	0.08
	Water, calcium, 1	0.94	0.07
	Water, calcium, 1×10^{-4}	0.98	0.07
	Ethyl alcohol	1.10	0.06
	Carbon tetrachloride	1.10	0.05
	Dry air	1.36	0.04
Illite	Water, sodium, 1	1.77	0.37
	Water, sodium, 1×10^{-3}	2.50	0.65
	Water, calcium, 1	1.51	0.28
	Water, calcium, 1×10^{-3}	1.59	0.31
	Ethyl alcohol	1.48	0.19
	Carbon tetrachloride	1.14	0.04
	Dry air	1.46	0.04
Smectite (montmorillonite)	Water, sodium, 1×10^{-1}	5.40	1.53
	Water, sodium, 5×10^{-4}	11.15	3.60
	Water, calcium, 1	1.84	0.26
	Water, calcium, 1×10^{-3}	2.18	0.34
	Ethyl alcohol	1.49	0.10
	Carbon tetrachloride	1.21	0.03
Muscovite	Water	2.19	0.42
	Carbon tetrachloride	1.98	0.35
	Dry air	2.29	0.41
Sand			0.01-0.03

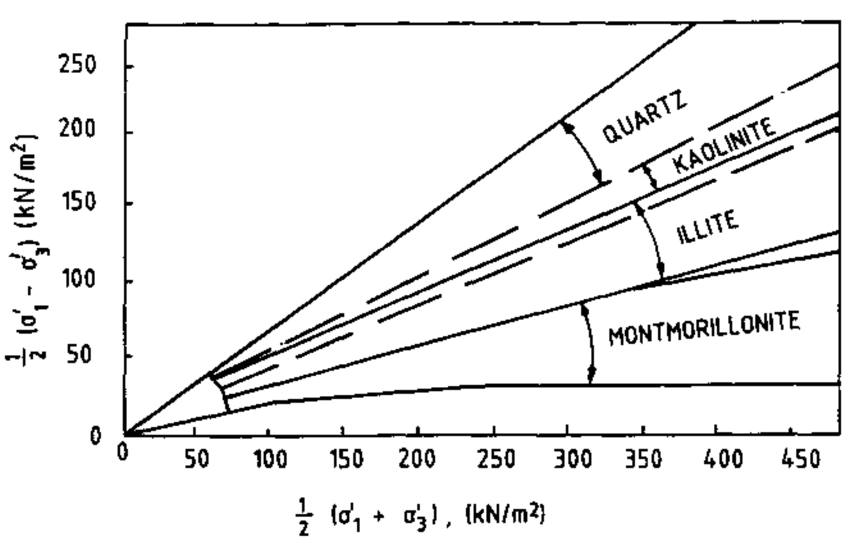


Figure 8.17. Ranges in effective stress failure envelopes for pure clay minerals and quartz (Mitchell 1976). Reprinted with permission of John Wiley & Sons Inc.

minerals, with varying electrolyte concentrations and for different cations in the electrolyte.

It can be seen that:

– exchange of Na^+ by Ca^{++} in montmorillonite (smectite) significantly reduces swell potential, but has only a minor effect on kaolinite.

– increase of concentration of Na^+ causes a marked decrease in the swell potential of montmorillonite, but no change in the swell potential of kaolinite.

8.5.3 *Shear strength*

The shear strength, parameters c', ϕ' (peak strength) and C'_R, ϕ'_R (residual strength) are dependent on the clay fraction percentage as shown in Figures 6.4 and 6.5. Clays with a high clay fraction percentage exhibit lower shear strengths and a greater reduction from peak to residual strength. As shown in Figures 6.5, 6.22 and 8.17 the shear strength is also dependent on the types of clay minerals present, with lowest strength for montmorillonites, highest for kaolinite.

8.6 IDENTIFICATION OF DISPERSIVE SOILS

8.6.1 *Laboratory tests*

There are several laboratory tests which can be used to determine the dispersivity of a soil.

8.6.1.1 *Emerson class number*

In this method, the soil is sieved through a 4.75 mm sieve and collected on a 2.36 mm sieve and tested as outlined in Figure 8.18. The test procedure is detailed in Standards Association of Australia (1980).

The test is carried out in distilled water, but may be repeated in water from the dam, or groundwater. This often gives significantly different results due to the presence of dissolved salts in the water (higher salt content gives less dispersive results). The soils are graded according to class, with Class 1 being the highly dispersive, Class 8 non dispersive. Soils with Emerson Class 1 to 4 need to be treated with caution in dam construction.

The Emerson test allows identification of dispersive soils, but does not provide a measure of their erodibility. It is inexpensive, and a useful first check on dispersivity.

8.6.1.2 *Soil Conservation Service test*

Soil Conservation Service test, also known as the double hydrometer test, or percent dispersion test (Standards Association of Australia 1980).

This involves two hydrometer tests on soil sieved through a 2.36 mm sieve. The hydrometer tests are carried out with dispersant, and without. The percent dispersion is

$$\frac{P}{Q} \times 100$$

where P = percentage of soil finer than 0.005 mm for the test without dispersant

Q = percentage of soil finer than 0.005 mm for the test with dispersant.

Sherard et al. (1976) indicate that soils with a percent dispersion greater than 50% are susceptible to dispersion and piping failure in dams, and those with a percent dispersion less than 15%

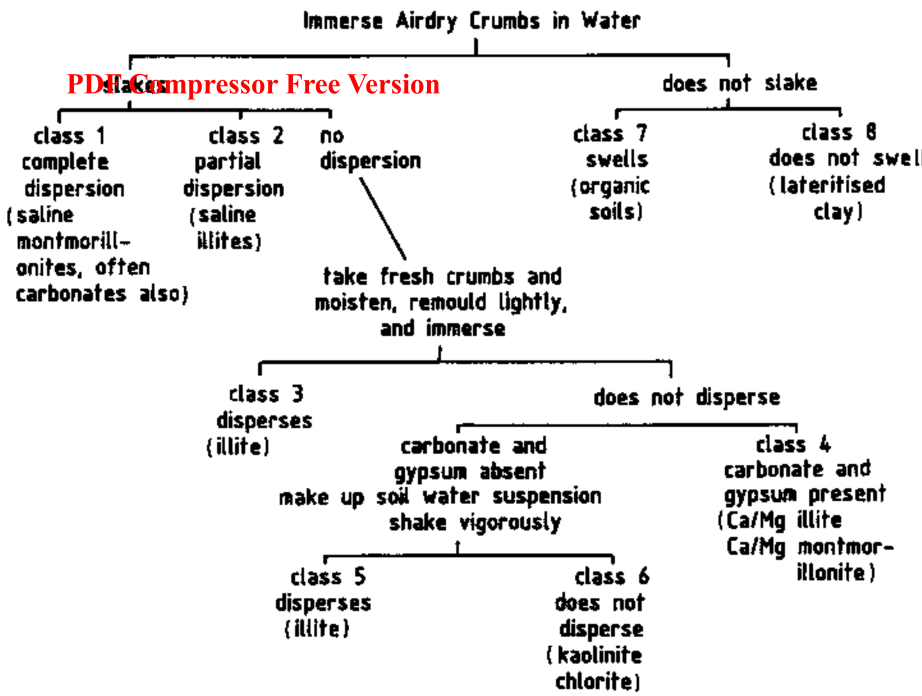


Figure 8.18. Determination of the Emerson class number of a soil (Ingles & Metcalf 1972).

are not susceptible. They also indicated good correlation between the percent dispersion test and the pinhole test described below.

8.6.1.3 *Pinhold dispersion classification*

Pinhold dispersion classification, also known as the pinhole test, or Sherard pinhole test (Standards Association of Australia 1980).

This test was developed by Sherard et al. (1976). A 1.0 mm dia hole is preformed in soil to be tested, and water passed through the hole under varying heads and for varying durations. The soil is sieved through a 2.36 mm sieve and compacted at approximately the plastic limit to a density ratio of 95% (to simulate conditions in a dam embankment with a crack or hole in the soil). Figures 8.19, 8.20 and Table 8.5 show the test arrangement, and criteria for classifying the soil.

Soils which tested as D1 and D2 were found by Sherard et al. (1976) to have suffered piping failure in earth dams, while those with ND1 and ND2 classification had not.

As for the Emerson class number, the results are dependent on the chemistry of the water used for the test (the standard test uses distilled water). The method is relatively simple with moderate cost, and has the advantage that it identifies soil erodibility directly, rather than indirectly. All dispersive soils are erodible to some degree – usually highly – but erodibility depends on other than clay fraction also.

8.6.1.4 *Chemical tests*

Based on correlation with many dam failures Sherard et al. (1976) proposed Figure 8.21 to

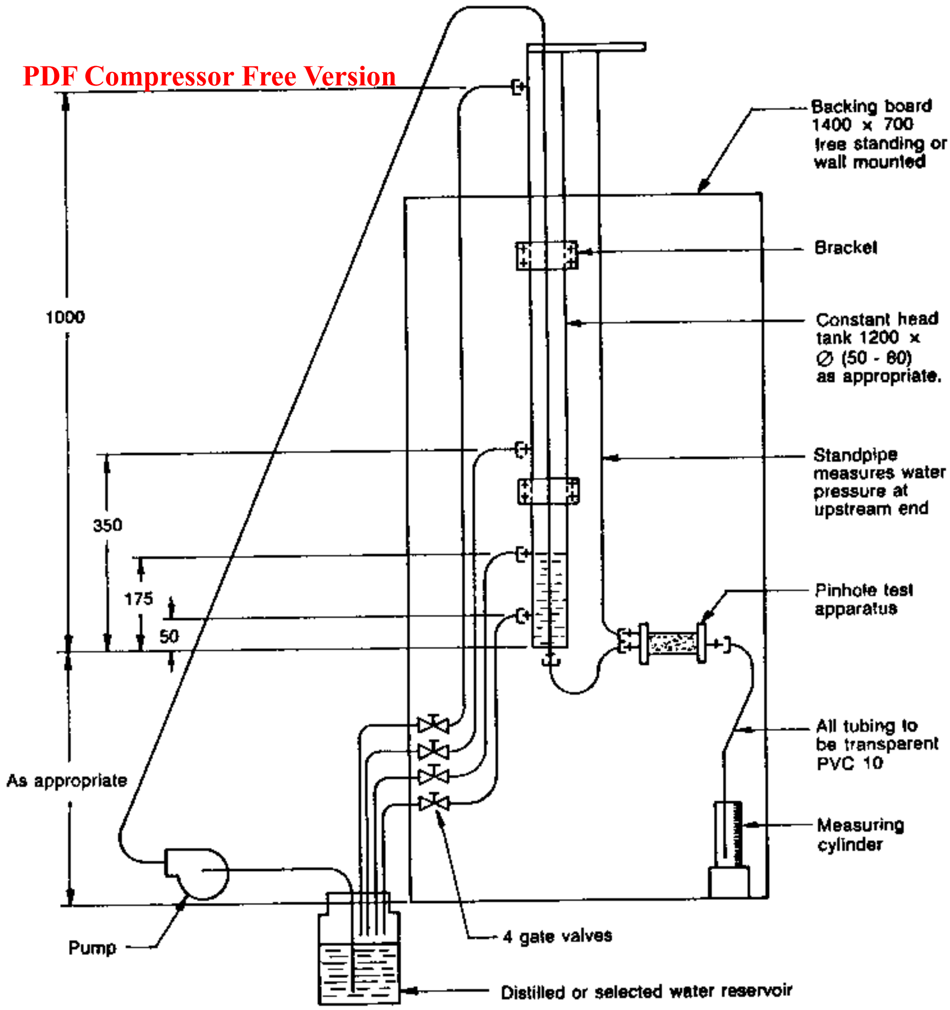


Figure 8.19. General layout of the pinhole dispersion apparatus (Standards Association of Australia 1980).

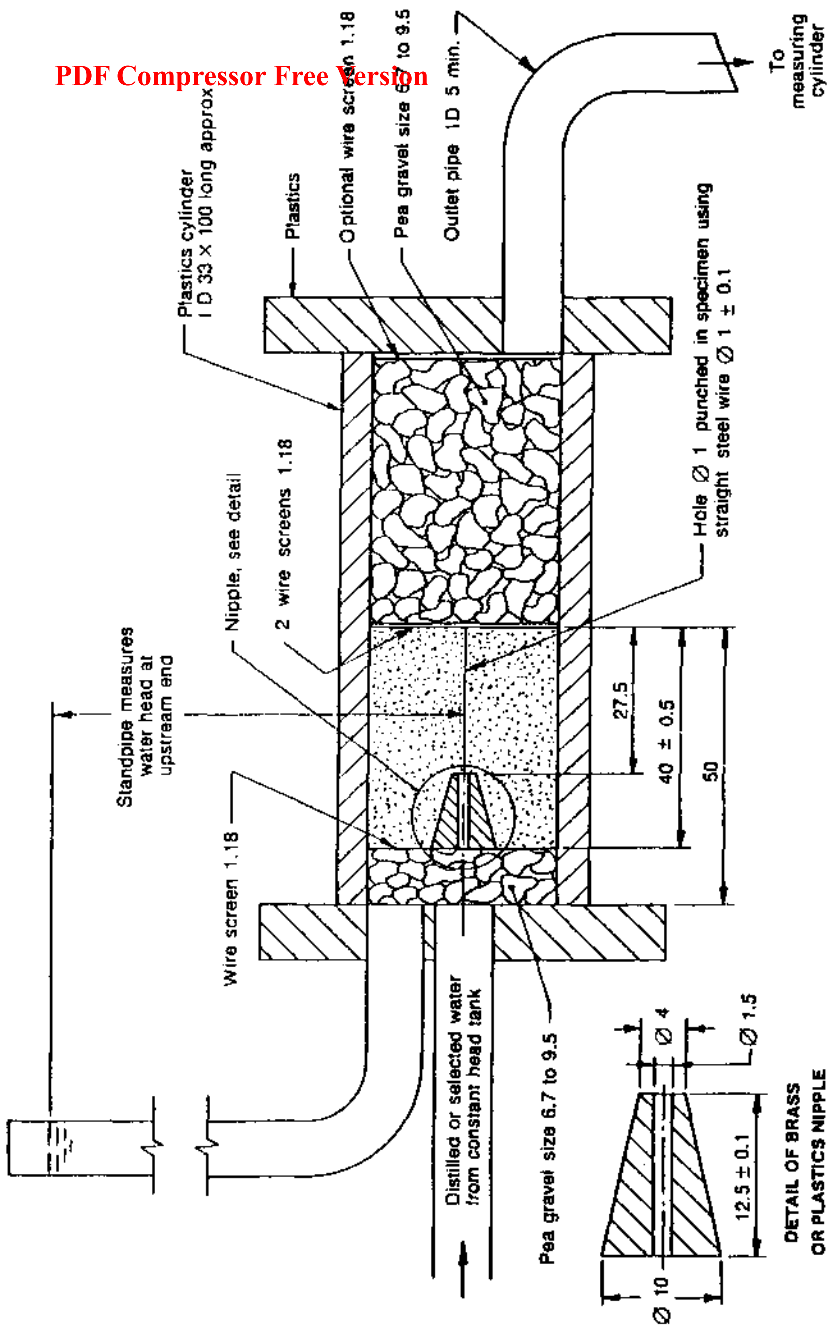
determine the dispersivity of soil. The sodium adsorption ratio (SAR) is as defined below and is determined by chemical test on the soil water.

Two terms which are useful for assessing the likely dispersivity of a soil are:

Sodium adsorption ratio (SAR)

$$= \frac{Na^{+}}{[\frac{1}{2}(Ca^{++} + Mg^{++})]^{1/2}}$$

in which Na^{+},Ca^{++}, and Mg^{++} are measured in milliequivalents per 1/2 litre



NOTE: Comparable arrangements are acceptable provided that the essential dimensions are maintained.

DIMENSIONS IN MILLIMETRES

Figure 8.20. Pinhole dispersion apparatus – soil specimen cylinder (Standards Association of Australia 1980).

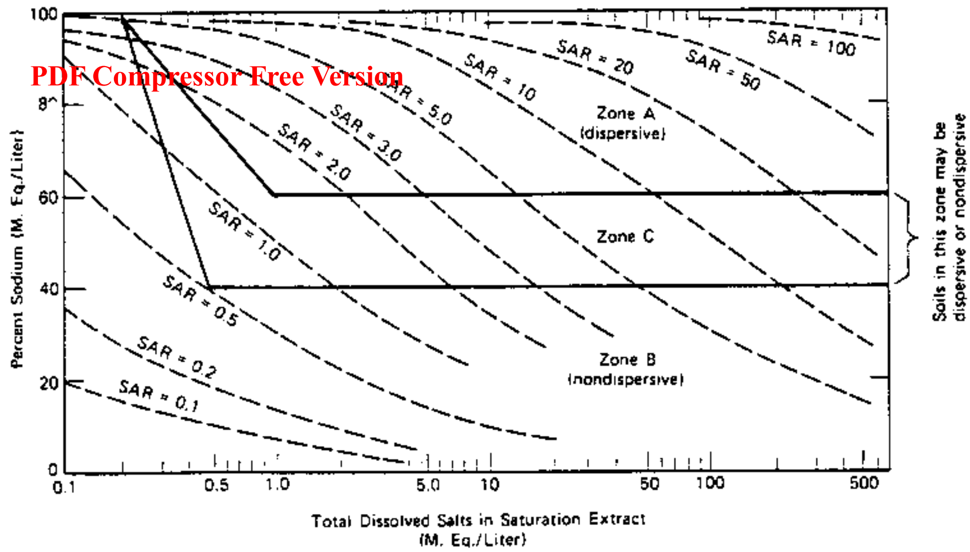


Figure 8.21. Relationship between dispersivity and dissolved pore water salts content (Sherard et al. 1976).

Table 8.5. Pinhole dispersion classification, criteria for evaluation of results (AS1289).

Classification		Head at termination of test (mm)	Test time of head (min)	Visibility of colour of flow at end of test	Final flow through specimen (ml/s)	Ratio of final/initial hole diameter after test to nearest 0.5
Designation	Description					
D1	Highly dispersive	50	10	Very distinct	> 1.5	≥ 2
D2	Dispersive	50	10	Distinct to slight	> 0.9	2.0
PD1	Potentially dispersive (intermediate)	50	10	Slightly but easily visible	< 0.9	1.5
PD2	Potentially dispersive (intermediate)	175 to 350	5	Slight	> 2.5	2.0
ND2	Non-dispersive	1000	5	Clear or barely visible	> 3.5	2.0
ND1	Completely erosion resistant	1000	5	Crystal clear	< 5.0	1.0

and

Exchangeable sodium percentage (ESP)

$$= \frac{Na^+}{\text{total exchange capacity}} \times 100$$

or

$$= \frac{Na^+}{Na^+ + K^+ + Ca^{++} + Mg^{++}} \times 100$$

For example, soils with an ESP greater than about 2% are susceptible to dispersion.

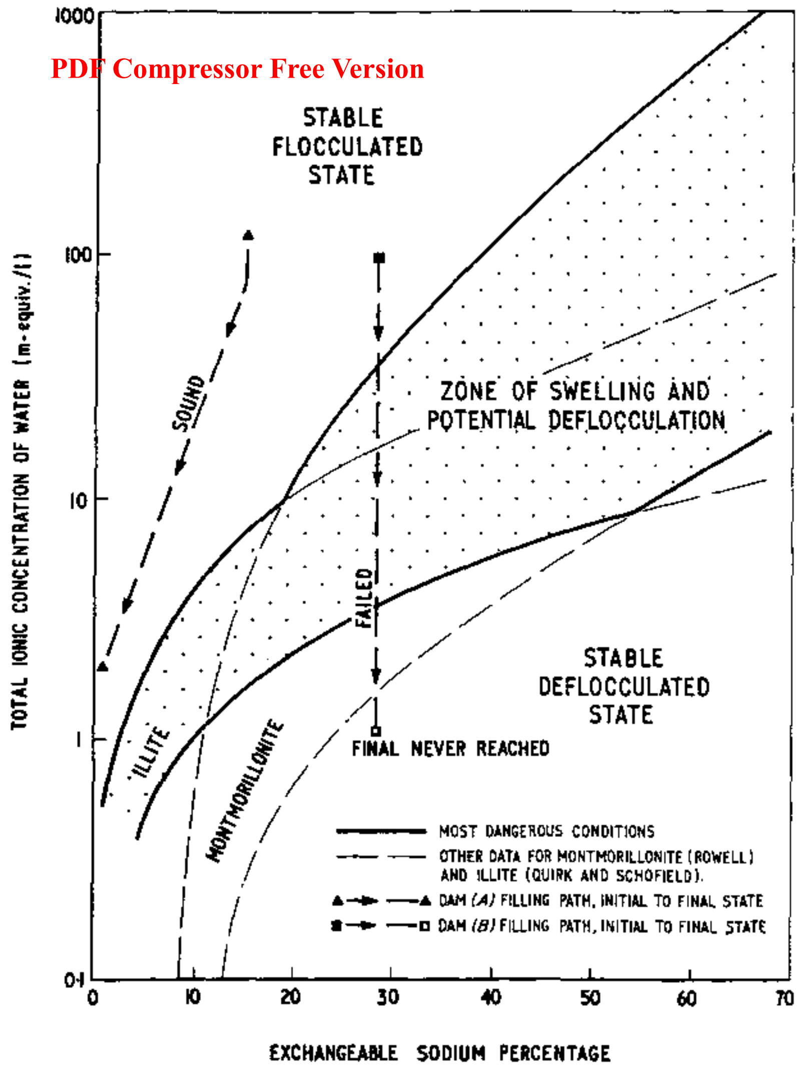


Figure 8.22. Estimation of dispersivity from exchangeable sodium percentage (Ingles & Metcalf 1972).

ESP and SAR are determined by chemical analysis of the soil pore water.

Ingles & Metcalf (1972) suggest that to assess dispersivity potential, the exchangeable sodium percentage and total ionic concentration of the pore water should be determined. Then for montmorillonite and illite clays, Figure 8.22 is used to determine whether the soil is in a flocculated, or dispersed (deflocculated) state, and whether a change in pore water chemistry during filling of the reservoir can lead to dispersion.

In this diagram, filling of the storage for Dam A maintains a stable flocculated state, but for Dam B, dispersion would occur. This can be readily related to the diffuse double layer concept, where a change in concentration of salts in the soil water can lead to a larger diffuse double layer, and a tendency to disperse (Dam B), but exchange of sodium cations by say calcium would

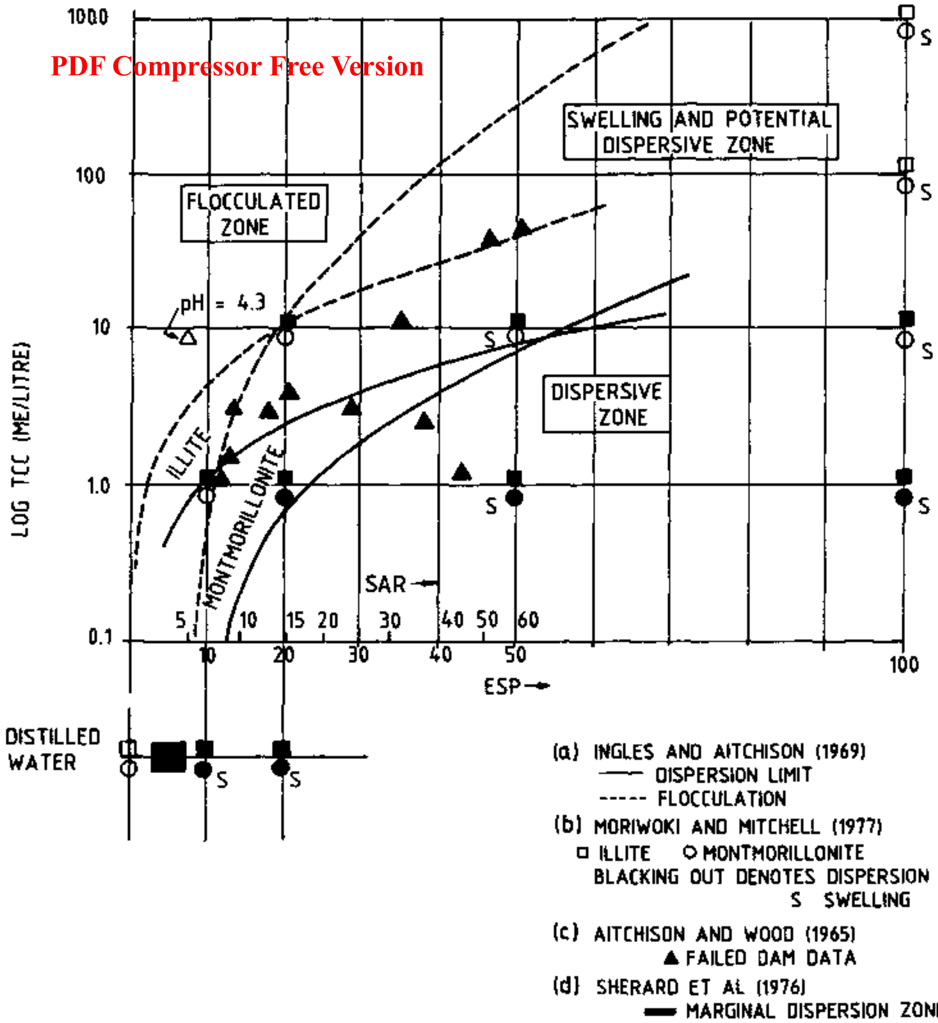


Figure 8.23. Estimation of dispersivity from exchangeable sodium percentage (McDonald et al. 1981).

retain a stable flocculated structure even though the total concentration was reduced (Dam A). McDonald et al. (1981) produced a revision of Figure 8.23 to include some work by Moriwaki & Mitchell (1977).

This indicated that the Ingles & Metcalf (1972) boundaries were reasonable except that dispersion could occur in montmorillonites at higher ESP than indicated by the earlier work. They suggested the use of the pinhole test for situations where water would be flowing (e.g. a crack in a dam) and Emerson test for quiescent conditions, e.g. reservoir. They also point out that the ESP can vary a lot on any dam and make use of cheaper tests such as Emerson and pinhole tests more attractive. It is also pointed out that where soils which are rich in bicarbonates are allowed to dry out, precipitation of relatively unsoluble $CaCO_3$ may occur, resulting in a lower Ca^{++} ion concentration and hence an increased potential for dispersion.

Most authors consider that it is necessary to use more than one test to ascertain the dispersivity of a soil. Sherard & Decker (1977) suggest that four tests should be used: Soil Conservation Service, pinhole, Emerson and chemical test. They were of the opinion that the pinhole test was best. Moore et al. (1985) indicate that in their experience:

– similar results were obtained using the pinhole and Emerson tests,

– the pinhole test was not particularly sensitive to compaction water content,

– only approximate agreement was found between the SCS test and the pinhole and Emerson tests,

– the Sherard et al. (1976) chemical test shown in Figure 8.22 showed a soil to be dispersive when it was known not to be (by its field behaviour and other tests).

The authors have used the Emerson class number and pinhole tests – usually in a ratio of 2 or 3 to 1, reflecting the greater cost of the pinhole test. If one accepts that the objective is only to identify potential problems this would seem an adequate approach. The chemical approach in Figure 8.23 has attractions in that it allows a prediction of behaviour with altered water quality and can be related to clay mineralogy. However, the Emerson class and pinhole tests can also be related to changes in water quality by carrying out these tests in water other than distilled water.

8.6.2 *Field identification and other factors*

While laboratory tests are a useful way of identifying dispersive soils, much can be determined by observing the behaviour of the soils in the field, e.g:

– The presence of deep erosion gullies and piping failure in existing small farm dams usually indicates the presence of dispersive soils, see Figures 8.24 and 8.1.

– Erosion of road cuttings, tunnel erosion along gully lines and erosion of weathered or clay

Figure 8.24. Deep erosion gullying in dispersive soils (Soil Conservation Service of NSW).

Table 8.6. Clay mineral identification from the environment (Ingles & Metcalf 1972).

Observation	Dominant clay component
Turbid water of strong yellow-brown to red-brown colour	Montmorillonites, illite plus soil salinity
Clear waters	Calcium, magnesium or ironrich soil, highly acid soil sands
Clear waters with a bluish cast	Non-saltine kaolins
Erosion gullies and/or field tunnelling in the natural soil	Saline clays, usually montmorillonites
As above, mild	Kaolinites
Landslips	Kaolinites, chlorites
Surface microrelief (gilgai)	Montmorillonites
Country rock type granitic	Kaolinites, micas
Country rock type basaltic, poorly drained topography	Montmorillonites
Country rock type basaltic, well drained topography	Kaolinites
Country rock type sandstones	Kaolinites
Country rock type mudstones and shales	Montmorillonites or illite, often soil salinity
Country rock type limestone	Alkaline montmorillonites and of very variable properties
Country rock type recent pyroclastics	Allophanes

Table 8.7. Clay mineral identification from the soil profile (Ingles & Metcalf 1972).

Inferences from the profile Observation	Dominant clay component
Mottled clays, red-orange-white mottle	Kaolinites
Mottled clays, yellow-orange-gray mottle	Montmorillonites
Medium to dark gray and black clays	Montmorillonites
Brown and red-brown clays	Appreciable illite, some montmorillonite
White and light gray clays	Kaolinites and bauxites
Discrete microparticles of high light reflectance (micas)	Micaceous soils
Discrete microcrystals, easily crushed	Gypsum-rich soils, or (rarer) zeolites
Soft nodules, acid-soluble, disseminated	Carbonates
Hard nodules, red-brown	Ironstones, laterite
Extensive cracking, wide, deep and closely spaced at 5 to 6 cms or less	Calcium-rich illites and montmorillonites
Up to intervals of 30 cms and more	Illites
Open-textured friable loamy soils with appreciable clay content	Usually associated with carbonate, allophane or kaoline, but never montmorillonite and seldom illite
Open-textured friable loamy soils with appreciable clay content, black	Organic soils, peats
Open-textured friable loamy soils of low clay content	Carbonate, silts and sands
Wormy appearance on exposed pre-existing weathered profile	Montmorillonites, plus soil salinity
Relatively thin, strongly bleached horizon near the soil surface (up to 60 cms from the top)	Above the bleach, fine silt: below the bleach, dispersive clay. Probably a seasonal perched water table at the bleach level

infilled rock joints may indicate potentially dispersive soils.

– The presence of cloudy water in farm dams and puddles of water after rain indicates dispersive soils.

One can infer the clay mineralogy from such observational techniques. Some guidelines are given in Tables 8.6 and 8.7 reproduced from Ingles & Metcalf (1972).

The geology of the area can also be a guide to dispersivity.

Sherard & Decker (1977) indicate that:

– Many dispersive clays are of alluvial origin. (The authors' experience is similar but there are many non dispersive alluvial clays. Some slopewash clays are also dispersive).

– Some soils derived from shales and claystones laid down in a marine environment are also dispersive.

– Soils derived from weathering of igneous and metamorphic rocks are almost all non dispersive (but may be erodible, e.g. silty sand derived from grandiorite).

– Soils with a high organic content are unlikely to be dispersive (this needs to be treated with caution, since many 'black cotton' soils are dispersive.

8.7 USE OF DISPERSIVE SOILS IN EMBANKMENT DAMS

8.7.1 *Problems with dispersive soils*

Dispersive soils are a major contributing factor to piping failure of embankment dams, particularly for small dams constructed without proper filters and often with poor construction supervision. However, failures do occur in structures which are reasonably well engineered. The main contributary factors are:

– the presence of dispersive soils in the embankment or foundation;

– poor compaction of the soil, i.e. to a low density ratio and/or dry of optimum water content;

– poor compaction of soil around pipes which pass through the embankment;

– the presence of soil structure, e.g. root holes, fissures or cracks, in the soil in the dam foundation, and with no adequate cutoff or erosion control measures;

– erosion of dispersive or erodible embankment soils into open fractures in the rock in the sides of the cutoff trench;

– poor cleanup of loose soil, grass etc from the cutoff foundation prior to placing earthfill;

– rapid filling of storages, giving insufficient time for cracks in the soil in the embankment to be sealed by the soil swelling or being wetted. The cracks may be due to desiccation during or after construction, differential settlement or hydraulic fracturing.

As indicated in ICOLD (1990) the majority of piping failure in dams constructed of dispersive soil occur on first filling, including cases when the reservoir has been raised after being at a given elevation for a period of time.

Piping failure can also be caused by introduction of water with low ionic concentration (i.e. 'fresh' water with a low salts content) into soils which were naturally saline, e.g. as shown in Figure 8.23. Ingles & Wood (1964) describe such a case. ICOLD (1990) indicates that virtually all failures occur in the embankment, not through soil in the foundation. However the authors have observed failures through the foundation in small farm dams. Examples of piping failure of dams constructed using dispersive soils are given in Aitchison, Ingles & Wood (1963), Aitchison & Wood (1965), Cole & Lewis (1960), Sherard & Decker (1977), Phillips (1977), Cole (1977), Rallings (1960), Wagener et al. (1981).

The authors' experience has been that piping failures usually occur in farm dams which have not been compacted other than by tracking of a bulldozer, but that even apparently well engineered structures have failed.

Examination of these cases shows failure to be related to the presence of dispersive soils (Emerson Class 1 or 2) compacted poorly (density ratio as low as 90% of standard compaction, as much as 2 to 4% below optimum water content, around outlet pipes through the embankment. In one case failure on first filling occurred despite the downstream slope of the dam being very flat (15H:1V).

A contributory factor also has been use of rough corrugated surfaces on the concrete surrounding the outlet pipe, precluding proper compaction of the soil. In another case, cracking of soil by drying during construction around the trench into which the outlet pipe was placed and concreted was a contributory factor (see Fig. 13.9).

8.7.2 *Construction with dispersive soils*

As indicated in ICOLD (1990), safe dams can be built with dispersive clay provided certain precautions are taken. ICOLD (1990) indicates that the concern about the problems of dispersive soils and attention to precautions increase with dam size. While the authors understand the sentiment that failure of a large dam may be more important than a smaller one, they would argue that in reality it is easier to build in the necessary precautions into larger dams than smaller. In most cases, normal good practice for high hazard dams will be all that is needed, e.g. well designed filters, proper foundation preparations and good construction control. In small dams such measures may be regarded as uneconomic.

The following outlines the main precautions which should be adopted. These are based on ICOLD (1990) and the authors' own experience.

8.7.2.1 *Provide properly designed and constructed filters*

Sherard et al. (1984a, b) have proven conclusively that erosion of dispersive soils can be controlled by properly designed filters. If the guidelines outlined in Chapter 7 are followed, and filters are provided:

– within the embankment to control erosion of the Zone 1 earthfill;

– on the foundation, if it is dispersive soil, either as a horizontal drain or as a filter layer between the foundation and rockfill;

– around outlet pipes, as shown in Figure 13.10.

Then there should not be problems with piping failure.

8.7.2.2 *Proper compaction of the soil*

Dispersive soils, particularly if being placed around outlet pipes, or at the contact between the earthfill and concrete structures, or if no filters are being provided (e.g. in a small low hazard dam), must be properly compacted or there is a high probability of piping failure.

ICOLD (1990) recommend compaction at a water content above optimum water content so as to avoid a flocculated soil structure, and to avoid brittleness which will promote formation of cracks. They suggest that a permeability of lower than 10^{-7}m/sec is required. This seems a high figure and one would normally expect to achieve 10^{-9}m/sec.

The authors' opinion is that a water content between optimum and optimum plus 2%, and a density ratio of greater than 98% (standard compaction) is desirable, and that the water content should not be below optimum -1%. If the soil was at optimum +2% or even say optimum +3%, one might relax the density ratio requirement to 97%, in recognition that the soil would probably be low permeability and less likely to crack than if compacted at optimum -1%.

This will necessitate use of thin layers, particularly adjacent outlet pipes, where rollers

cannot be used. Supervision and testing need to be very thorough. Care must be taken to avoid drying of the surface of layers of earthfill, which could result in cracking, and a preferred path for piping failure to initiate.

8.7.2.3 *Careful detailing of pipes through the embankment*

Pipes through embankments should be avoided if possible since it is very difficult to ensure good compaction around the pipe, and differential settlement can also lead to cracking of the soil. If pipes must be placed through an embankment constructed by dispersive soil then the following alternatives should be considered:

Either (a) support the pipe on a concrete footing and use filters to surround the downstream end of the pipe as shown in Figure 13.11; or (b) support the pipe on a concrete footing and modify the dispersive soil with lime to render it less dispersive as detailed below; or both (a) and (b).

It is considered that reliance on good compaction without the filter or lime modification is a relatively high risk option and is not recommended for highly dispersive soils.

8.7.2.4 *Lime or gypsum modification of the soil*

Most dispersive soils can be rendered non dispersive by addition of a small quantity of lime [$Ca(OH)_2$ or CaO] or gypsum [$CaSO_4$ or $CaSO_4(2H_2O)$].

This process is one of cation exchange, with the Ca^{++} ions exchangeable for Na^+ ions. Laboratory tests can be carried out to determine the required amount of lime or gypsum, e.g. Sherard pinhole tests using soil with different amounts of lime or gypsum. Commonly one would require 2 to 3% lime, and would allow a margin of 1 or 2% above that indicated by the laboratory tests to allow for difficulty in mixing the lime with the soil. Ideally the lime would be mixed with the soil using a pulveriser. This breaks up the soil so that 80 to 90% of the particles are less than 25 mm diameter.

8.7.2.5 *Sealing of cracks in the abutment and cutoff trench*

If the soil used for construction is dispersive, particular care must be taken to seal cracks in the cutoff foundation, and the sides of the cutoff trench so the soil will not erode into the cracks. Extensive use of slush concrete or shotcrete is likely. If the cutoff trench is in soil, it may be necessary to provide a filter on the downstream side as shown in Figure 9.7.

8.7.3 *Turbidity of reservoir water*

The presence of dispersive soil in the reservoir area, or in the catchment, can lead to turbidity of the water in the reservoirs. Grant et al. (1976) describe how turbidity in the water for Cardinia Creek Dam, Melbourne, was prevented by adding 6500 tonnes of gypsum to the water on first filling, at a concentration of 40 mg/l of water. Again the process is one of Ca^{++} displacing the Na^+ in the soil and makes it less dispersive. Gypsum was added to water as it entered the storage. Cardinia Creek Dam was off river.

McDonald et al. (1981) describe how turbidity in Ben Boyd Dam, Eden NSW, was controlled by adding 1% by weight of gypsum of the soil depth in the reservoir area (27 tonnes per hectare). Gypsum was mixed in to 150 mm depth by disc harrow and compacted with a sheep's foot roller and gave initially clear water, but flood rains washed soil from the catchment and led to turbidity. The storage water was subsequently flocculated and clarified by adding 40 mg/l of aluminium $Al_2(SO_4)_3$ and 20 mg/l NaOH.

CHAPTER 9

Dams on highly permeable soil foundations

9.1 GENERAL DESCRIPTION OF THE SPECIAL PROBLEMS

When dams are constructed on highly permeable soil foundations e.g.:

– alluvial sands and gravels,

– colluvial sands and gravels,

– lateritic soils.

There are some common features of the foundations which necessitate the use of design features which are shown in Figure 9.3 to control underseepage and erosion.

Some of these features, in alluvial foundations, are shown schematically in Figure 9.1. They include:

A. Lenses or layers of lower permeability sand, silty sand, or even clayey sand may occur, giving a very much reduced vertical permeability. These may be present in point bar deposits.

B. There is often a coarser gravel, or even boulder/gravel layer at the base of the alluvium (channel lag deposit), reflecting the time when the river was more active. This may be very permeable.

C. The upper part of the rock surface may be permeable because of the presence of open joints, potholing and destressing. The surface may also be very irregular.

D. The coarse alluvium – sand/gravel.is in itself likely to be layered giving a high permeability ratio k_H/k_V, e.g. point bar deposits.

E. There is often a layer of silty sand/silty sandy gravel on the surface, giving a low permeability, e.g. in flood plain deposits.

In general these features combine to give a much greater horizontal than vertical permeability. Hence while water can enter into the foundation in the large area of exposed alluvium in the reservoir, it is inhibited from flowing into the horizontal drain. Similar layering can occur in soils of colluvial origin.

Figure 9.2 shows some common features of lateritised soil profiles which have developed by weathering of rock. The features include:

– Very high permeability upper zone of pisolithic gravel.

– Variably cemented and permeable ironstone zone.

– Mottled zone and underlying kaolin rich zone. This is commonly medium to high permeability due to relic joints, and root holes. Many of these features are near vertical and not intercepted by vertical boreholes. The mass permeability of the clay will commonly be of the order of 10^{-6} to 10^{-5} m/sec, i.e. the permeability of sand.

– The quartz rich zone overlying weathered bedrock is often a mixture of clean sand and

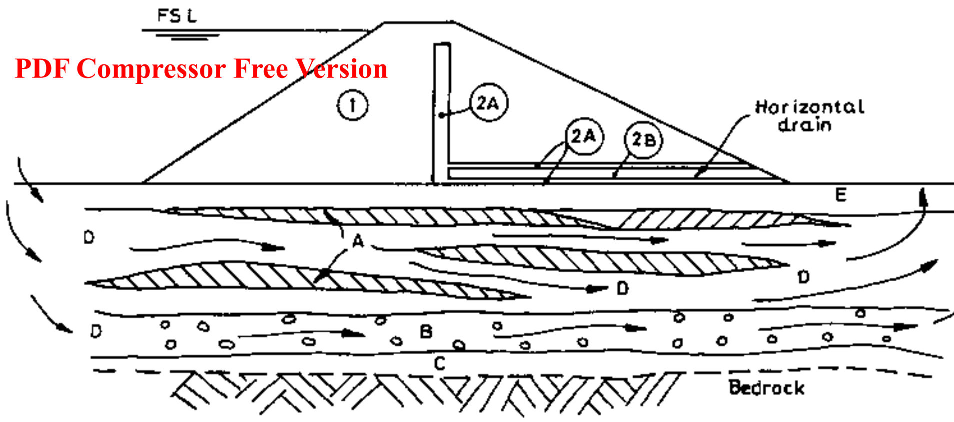


Figure 9.1. Some common features of alluvial foundations.

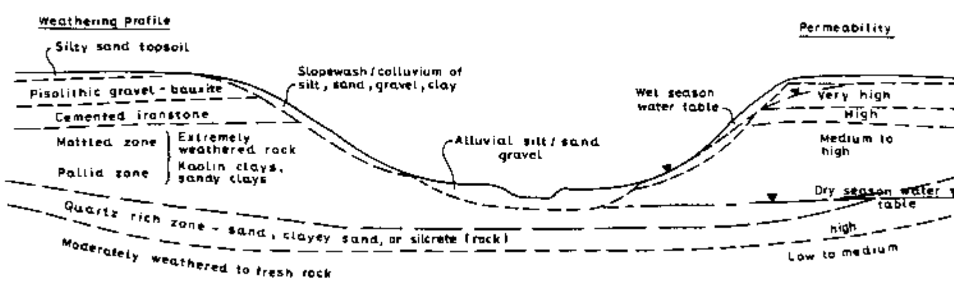


Figure 9.2. Some common features of lateritic foundations.

clayey sand of high permeability. In some cases it may be cemented as silcrete, but will still be highly permeable.

– The upper bedrock is often fractured and moderate permeability.

– The depth of weathering may be up to 30 m, e.g. in the Darling Ranges of Western Australia (Gordon 1984) and in the subtropical monsoonal climate areas of Weipa, North Queensland and Ranger and Jabiluka, Northern Territory. The weathering profile usually does not follow the topography but rises more gradually in the abutments as shown in Figure 9.2. At Weipa, Ranger and Jabiluka with low relief, it is the authors' experience that the depth of weathering is related to rock type rather than topography.

– The water table fluctuates markedly between end of wet season and end of dry season.

These conditions lead to some basic problems which have to be addressed in the design and construction of embankment dams. The problems include:

– limiting seepage quantities to acceptable levels,

– preventing erosion and piping failure through the foundation,

– maintenance of embankment stability including the effect on a soil strength foundation and seepage pore pressures.

These problems are overcome on a case by case basis. However, solutions often include one or more of the design features shown on Figure 9.3.

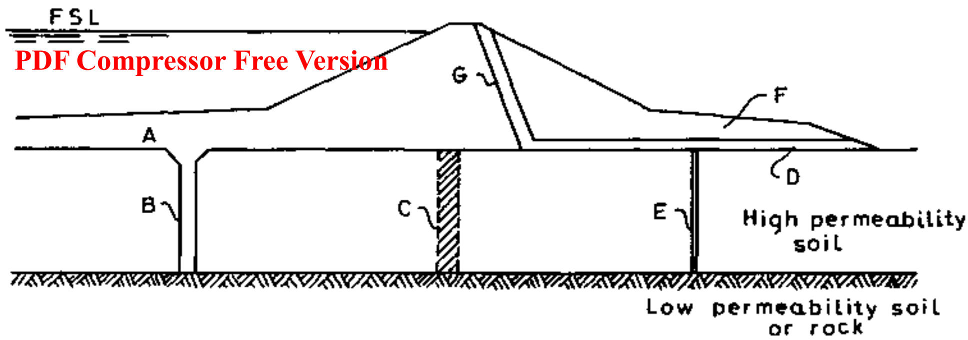


Figure 9.3. Design features for controlling seepage and erosion beneath earth dams on highly permeable foundations 1) Seepage-reducing methods: A, upstream impervious blanket; B, slurry trench; C, grouted zone. 2) Drainage methods: D, horizontal drain; E, relief wells; 3) Stabilizing methods: F, weighting berm; G, chimney drain.

The design features are:

A. Upstream impervious blanket which increases the seepage path, reduces seepage, and seepage exit gradients.

B. Slurry trench (or other types of cutoffs) which will reduce (or almost stop if fully penetrating to an impervious base) seepage and seepage exit gradients.

C. Grouting – an alternative to slurry trench.

D. Horizontal drain – which controls exit gradients and pore pressures, but actually increases the amount of seepage.

E. Pressure relief wells – control exit gradients where confined aquifers in the foundation reduce the effectiveness of the horizontal drain.

F. Weighting berm – which improves downstream slope stability and overcomes potential liquefaction or 'blowup' of the foundation (with E).

G. Chimney drain, which intercepts seepage through the dam and controls internal erosion and pore pressures.

9.2 CONTROL OF EROSION AND 'BLOWUP' OR LIQUEFACTION OF THE FOUNDATION

9.2.1 *Erosion control*

Seepage beneath a dam on a permeable soil (or permeable weathered rock) foundation should be allowed to exit in a controlled manner into a horizontal drain. The horizontal drain may consist of a single layer of Zone 2A filter, or 3 layers of Zone 2A and 2B as shown in Figure 9.1. In both cases Zone 2A should be designed to act as a filter to control erosion of the soil from the foundation into the drain. The filter design criteria detailed in Chapter 7 should be used.

The 3 layer drain incorporates the layer of high permeability Zone 2B to ensure that the drain has sufficient discharge capacity.

As detailed in Cedergren (1972) it is good practice to design the horizontal drain to have sufficient capacity to discharge the flow entering the drain from the dam foundation and from the vertical drain without the phreatic surface rising into the low permeability fill (see Fig. 9.4).

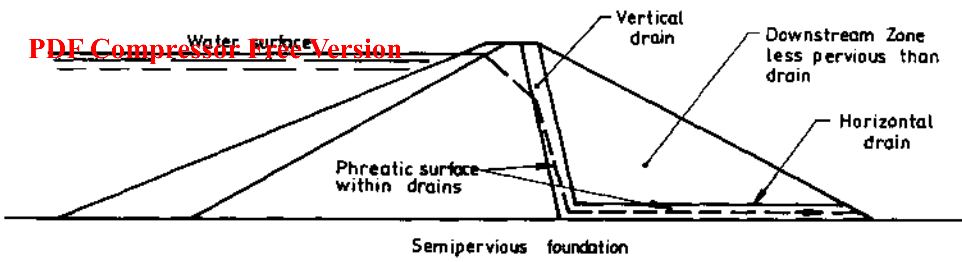


Figure 9.4. Earth dam with internal drain designed to prevent the phreatic surface from rising above the top of the drain (Cedergren 1972). Reprinted with permission of John Wiley & Sons Inc.

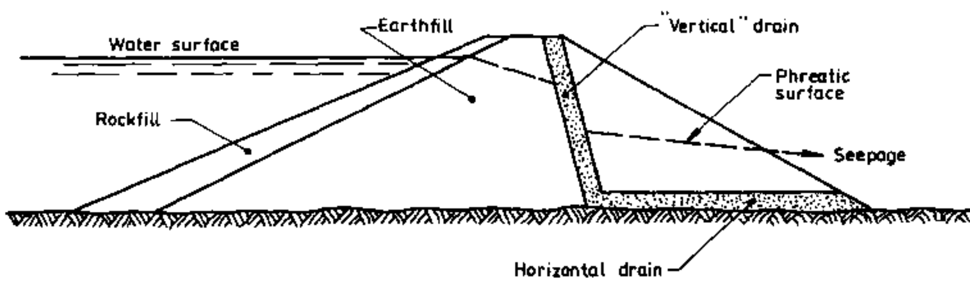


Figure 9.5. Earth dam with internal drains of inadequate discharge capacity.

If the horizontal drain has insufficient capacity, the phreatic surface will rise into the downstream low permeability fill as shown in Figure 9.5, reducing the stability of the downstream slope and also potentially leading to piping failure in the downstream fill.

Such lack of drain capacity can also occur on permeable rock foundations where earthfill dams such as those shown on Figures 9.4 and 9.5 are used. It can also occur where dirty rockfill has been used as the downstream zone.

Cedergren (1972) gives a design method for estimating the discharge capacity of a horizontal drain without pressurization based on:

$$q = \frac{kh^2}{2L}$$

where k = permeability of the drain material
h = vertical thickness of the drain
L = length of the drain
q = discharge capacity per metre width of drain (width measured across river).

The capacity of the vertical drain is seldom a critical issue because the quantity of seepage through the earthfill is small and the vertical drain width is dictated by construction factors. However its capacity should be checked by

$$q_2 = \frac{k_2 h_2 \cdot w}{L_2}$$

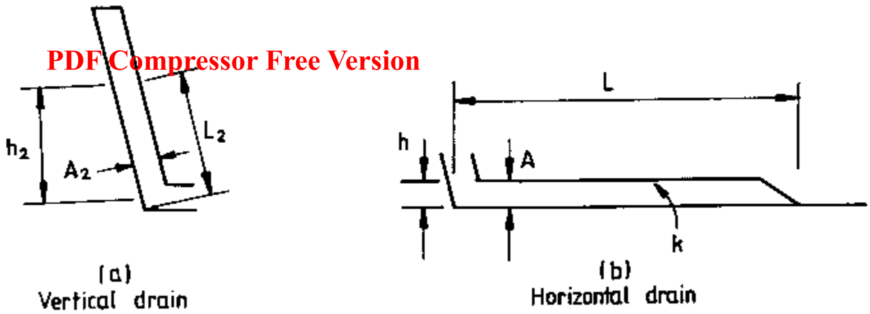


Figure 9.6. Design of drain dimensions for discharge capacity.

where k_2 = permeability of vertical drain h_2, L_2 as shown in Figure 9.6
  w = width of drain

When checking the design of the horizontal drain the following should be noted:

– Water from the abutments will flow towards the lowest part of the drain before flowing out the toe of the dam. Hence the required flow capacity per unit width in this area will be greater than the average flow per unit width under the dam.

– If a 3 layer filter drain is used, Zone 2B will dominate the discharge capacity.

– Conservative estimates of foundation permeability should be used for the design of the horizontal drain, since failure to provide adequate capacity can lead to failure of the dam, or the requirement for a stabilizing berm if the problem is recognised.

– Similarly, conservative estimates of the horizontal drain permeability should be used, and care taken to ensure the drain is not contaminated by soil from the foundation during construction.

– The downstream toe should be designed to ensure the drain outlet is not blocked by erosion of soil off the embankment or the abutment.

It is wise to provide a berm at the toe of the dam above the drain to collect any material eroded off the downstream face of the dam.

Another factor which should be considered is possible erosion of the earth core into the permeable soil downstream of a cutoff excavated into the permeable soil.

This may occur where coarse sand or gravel layers are present in alluvial soils, or open fissures or holes in lateritic soils. It can also occur with a cutoff excavated into columnar volcanics, pyroclastic soils or any open jointed rock. Generally erosion can be prevented by placing a filter zone between the earthfill and foundation, but it may be necessary to place concrete over open lateritic features.

It is critical that the horizontal drain be placed on the permeable foundation soil, so underseepage can be encouraged to enter the drain. This may involve removing low permeability soil as shown in Figure 9.7 where the sandy clay and silty sand has been removed from beneath the horizontal drain.

9.2.2 *Prevention of 'blowup,' 'boiling' or 'liquefaction' of the foundation*

If seepage water emerges at too high a gradient from downstream of the toe of a dam on alluvial foundations, 'blowup,' 'boiling' or 'liquefaction' of the toe can occur with the possibility of

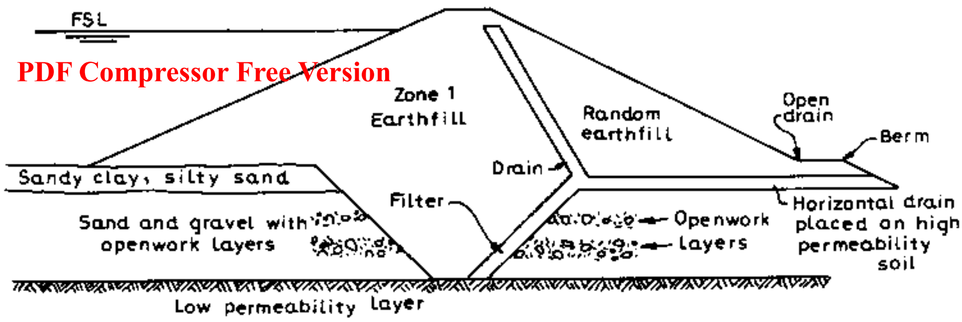


Figure 9.7. Control of erosion of earthfill into permeable foundation.

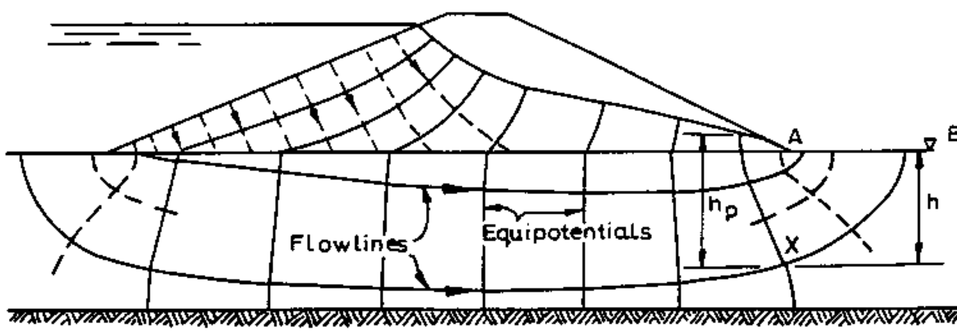


Figure 9.8. Locations downstream of a homogeneous dam where blowup may occur.

– loss of shear strength,
– potential for development of piping failure.

Figure 9.8 shows the area (A-B) which is susceptible.

The mechanism involved is one of the high pore pressure in the foundation leading to low effective stresses, with 'liquefaction' or blowup occurring when the effective stress becomes zero.

The factor of safety (F_{UT}) against this occurring can be calculated in two ways, the first being:

$$F_{UT} = \frac{\sigma_v}{u}$$

where σ_v = total vertical stress at any point in the foundation
u = pore pressure at the same point

For point X in Figure 9.8

$$F_{UT} = \frac{h\,\gamma_{SAT}}{h_p\,\gamma_w}$$

where γ_{SAT} = unit weight of saturated foundation soil

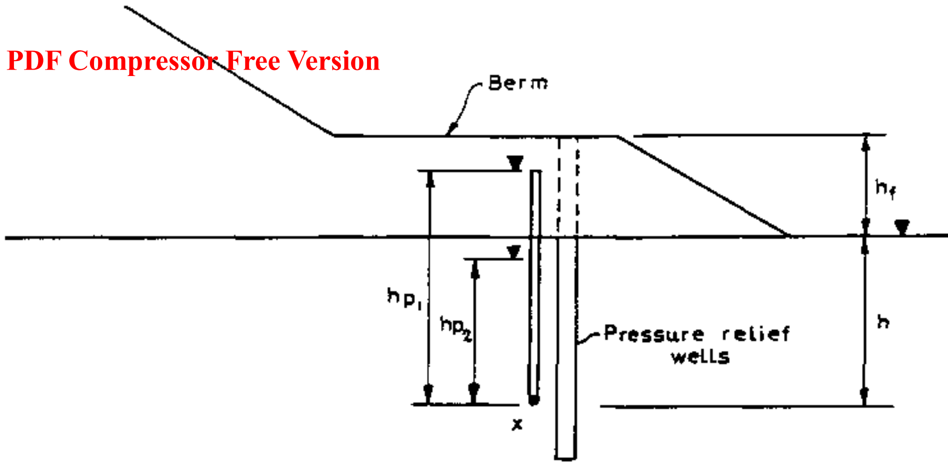


Figure 9.9. Control of liquefaction by pressure relief wells and weighting berm.

γ_w = unit weight of water

The factor of safety against blowup calculated in this manner should be compatible with the consequences of failure and degree of certainty of predicting pressures. However it should be at least 1.5, with pore pressures calculated conservatively (Cedergren 1972).

The alternative method for estimating the factor of safety is to consider the gradient of the flownet. If the gradient approaches unity, liquefaction can be expected to occur. The required factor of safety, i.e.

$$F_{UG} = \frac{\text{critical gradient}}{\text{actual gradient}}$$

$$= \frac{1}{\text{critical gradient}}$$

should be at least 3 and preferably 5.

The authors' preference is to use the F_{UT} definition, because this allows a better understanding of the ways in which the factor of safety can be improved.

In the event that pore pressures are calculated or known from piezometers to be too high, the factor of safety can be increased by adding a weighting berm (see Figure 9.9) or by providing pressure relief wells.

The weighting berm is effective by adding to the vertical stress, the pressure relief drains by reducing the pore pressure.

If the original pore pressure at X was hp_1, the original factor of safety would have been

$$F_{UF} = \frac{\gamma_{SAT}\ h}{\gamma_w\ hp_1}$$

If the berm of height hf and unit weight γ_f was constructed, then

$$F_{UF} = \frac{\gamma_{SAT}\ h + \gamma_f\ hf}{\gamma_w\ hp_1}$$

If no berm was provided but instead pressure relief wells were constructed, resulting in a reduction of pore pressure to γ_w hp$_2$ then

$$F_{UW} = \frac{\gamma_{SAT}\ h}{\gamma_w\ hp_2}$$

If both the berm and pressure relief wells were constructed

$$F_{UWF} = \frac{\gamma_{SAT}\ h + \gamma_f\ hf}{\gamma_w\ hp_2}$$

The following should be noted

– It is difficult to predict pore pressures near the toe of a dam, because they are greatly affected by local variations in permeability (see for example Figures 10.4, 10.5 and 10.6). Hence it is necessary to be conservative, and to provide piezometers to check the assumptions.

– The berm will be most effective if it is free draining.

– The pressure relief wells will provide greatest benefit at each well, and least benefit midway between wells. For conservatism the design should be based on the pore pressures midway between wells. Again, piezometers are required to check the effectiveness of the wells.

9.3 CONTROL OF UNDERSEEPAGE BY CUTOFFS

9.3.1 *General effectiveness of cutoffs*

As shown in Figure 9.3, seepage through a permeable foundation can be reduced by constructing a low permeability cutoff through the permeable material. This may consist of

- cutoff trench filled with earthfill,
- slurry trench,
- concrete diaphragm wall,
- contiguous or intersecting bored piles,
- sheet pile wall,
- grout curtain.

Such cutoffs will have varying degrees of effectiveness depending on their permeability and the depth to which they are taken. This is illustrated in Figures 9.10 and 9.11 which are reproduced from Cedergren (1972).

Figure 9.10 shows the effect on the line of seepage in the downstream shell of an embankment, and the effect on exit gradients, of cutoffs constructed to varying proportions of the total depth of a permeable foundation (note the example assumes the foundation and downstream fill zone have the same permeability).

It can be seen that there is not a significant improvement even with a cutoff which is 90% penetrating. Only where the cutoff is fully penetrating and properly connected into the low permeability zone will the seepage pressures be controlled. This is often difficult to achieve as will be discussed below.

Figure 9.11 shows the effect of fully penetrating grouted cutoffs on the line of seepage in the downstream zone. Unless the grouted zone (or cutoff constructed by another method) has a permeability much less than the foundation, there is little reduction in downstream pore pressures.

The effectiveness of partially penetrating cutoffs will depend on layering of lower and

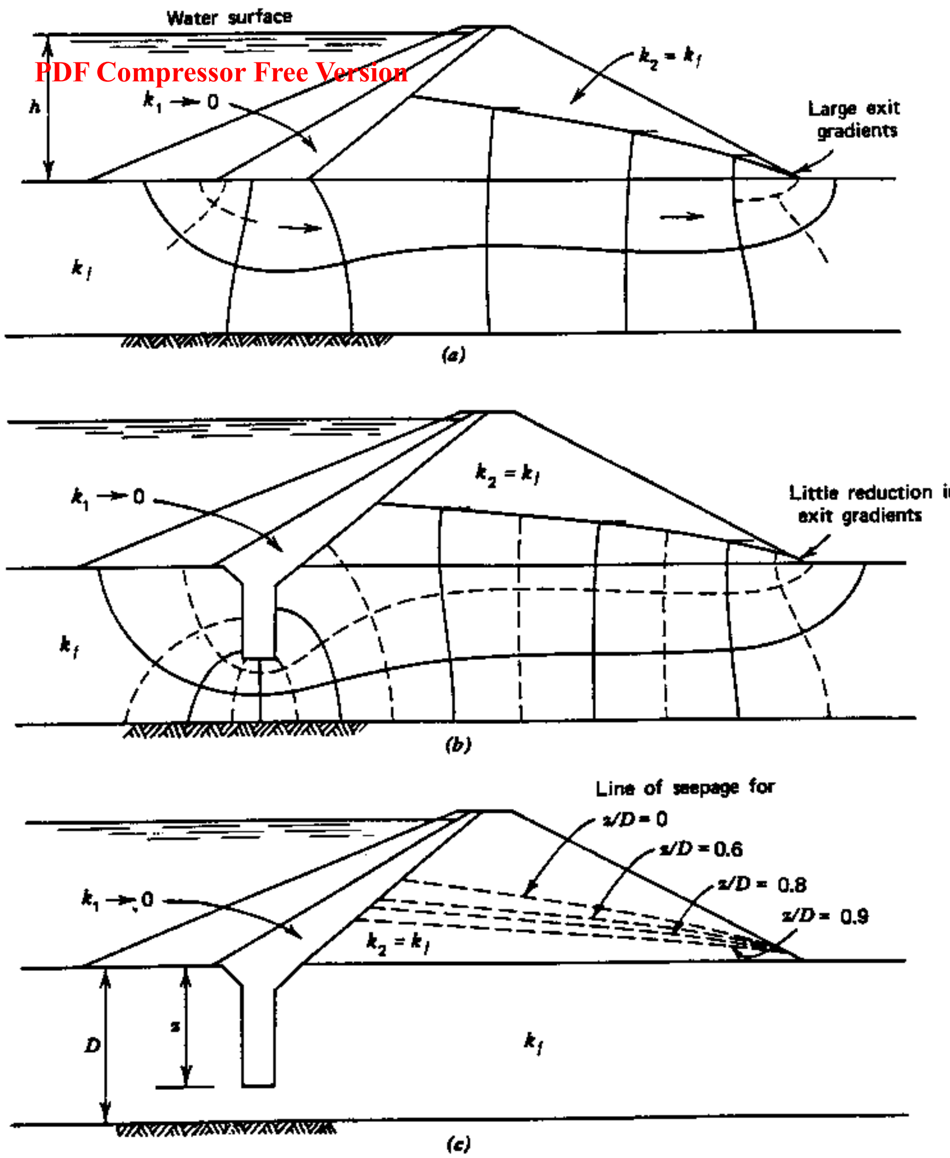


Figure 9.10. Effect of partial cutoff on position of line of seepage: a) flownets, z/D = 0. b) flownet, z/D = 0.6. c) position of line of seepage to various values of z/D (Cedergren 1972). Reproduced by permission of John Wiley & Sons Inc.

higher permeability soils in the foundation. If there are continuous low permeability layers present, partially penetrating cutoffs can be effective. However, it must be expected that unless a cutoff is fully penetrating, and of low permeability (say 10 to 100 times less than the permeable foundation) it will have little benefit in the reduction of leakage and exit gradients. This is discussed further in Chapter 12 (Section 12.2.4.6).

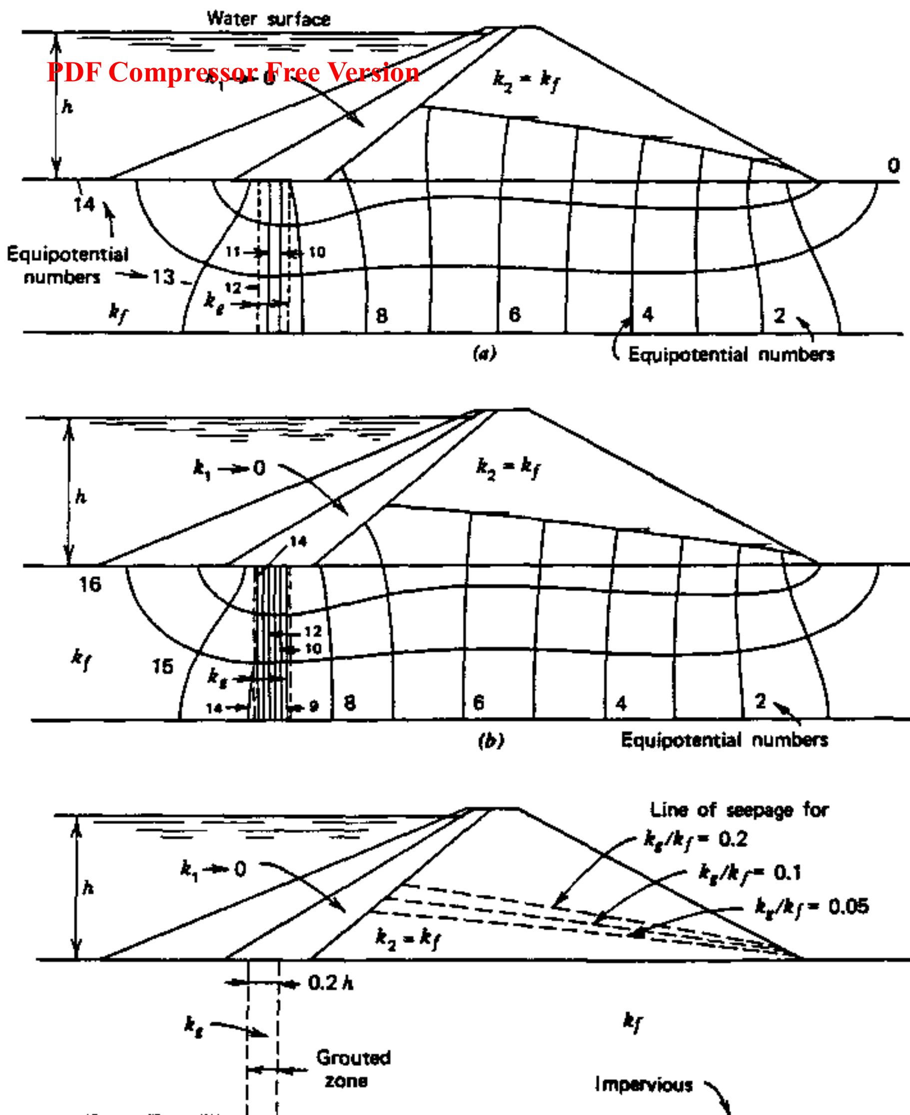


Figure 9.11. Effect of grouted cutoffs on position of line of seepage: a) flownet, $k_g = 0.2k_f$, b) flownet, $k_g = 0.1k_f$, c) position of line of seepage for various values of k_g/k_f (Cedergren 1972). Reproduced by permission of John Wiley & Sons Inc.

9.3.2 *Cutoff trench*

When the depth of permeable soil is relatively small, an effective way of providing a cutoff is to excavate a trench through the permeable layer and backfill with Zone 1 earthfill as shown in Figure 9.7. If there are continuous relatively low permeability layers within the soil the cutoff trench may be stopped at such a layer rather than penetrating to rock.

The practicality and economics of constructing the cutoff this way depend on:

– whether dewatering is necessary to construct the cutoff,

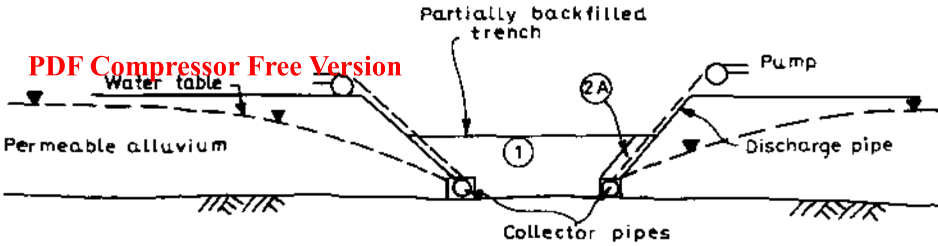


Figure 9.12. Dewatering system for cutoff trench.

– the availability of equipment to construct cutoffs by other methods, e.g. slurry trench,

– the stability of the sides of the trench during construction (which is dependent on soil type, consistency and effectiveness of dewatering).

In most cases the economic depth is likely to be less than 10 m. Beyond this depth slurry trench and other wall cutoffs are likely to be more economic.

If dewatering of the permeable soil is necessary, it is likely that the dewatering has to continue while the trench is being backfilled with earthfill. Figure 9.12 shows a possible dewatering arrangement.

As shown in Figure 9.7, a filter zone may have to be incorporated into the downstream side of the cutoff trench to prevent erosion of the earthfill into the foundation.

If a cutoff trench can be constructed it does provide a very good quality cutoff, with a low permeability. For compacted clayey earthfill it would not be unreasonable to expect a permeability of 10^{-8} to 10^{-9} m/sec. The contact with the foundation can be of high quality, and for rock foundation, grouting can be carried out if necessary from the base of the trench.

9.3.3 *Slurry trench cutoff*

Slurry trench cutoffs consist of a continuous trench excavated by means of a backhoe, dragline or clamshell, or combination thereof, with the trench supported by bentonite slurry.

The trench is backfilled with sand or sand and gravel thoroughly mixed with the slurry. Often the sand and gravel will be from the excavation. Figures 9.13, 9.14 and 9.15 show the technique.

The slurry trench is constructed 1 to 3m wide. The maximum depth depends on the method of excavation. ICOLD (1985) and Xanthakos (1979) indicate that practical maximum depths are:

– backhoe (excavator), 10m,

– dragline, 25 m maximum ; 20 m preferred maximum,

– dragline and clamshell, 25 m for slurry trench cutoffs.

Note that it may be difficult to form a low permeability connection between the cutoff and the underlying low permeability stratum, particularly if the surface is irregular. An air lift or clamshell, or scraper should be used as shown in Figures 9.14 and 9.15 to assist in cleanup of loosened debris at the base of the trench.

ICOLD (1985) indicates that the backfill will normally consist of bentonite slurry, with 5-15% bentonite (by weight), a marsh funnel viscosity of greater than 40 seconds, mixed with well graded sand and gravel between 0.02 and 30 mm size. The mixture should have a standard concrete cone slump of 100-200 mm. Xanthakos (1979) indicates that naturally occurring clays

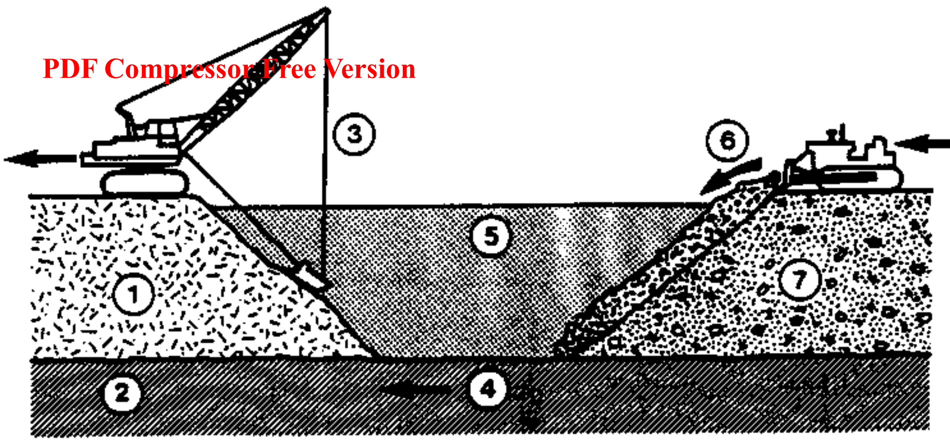


Figure 9.13. Slurry trench technique 1) Permeable virgin ground, 2) Substratum, 3) Excavation, 4) Direction of progress, 5) Trench filled with bentonite slurry, 6) Filling with aggregate, 7) Finished trench (ICOLD 1985).

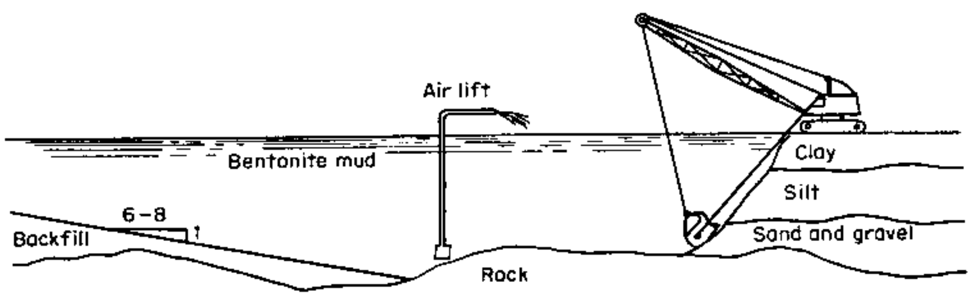


Figure 9.14. Slurry trench excavation by dragline (Xanthakos 1979).

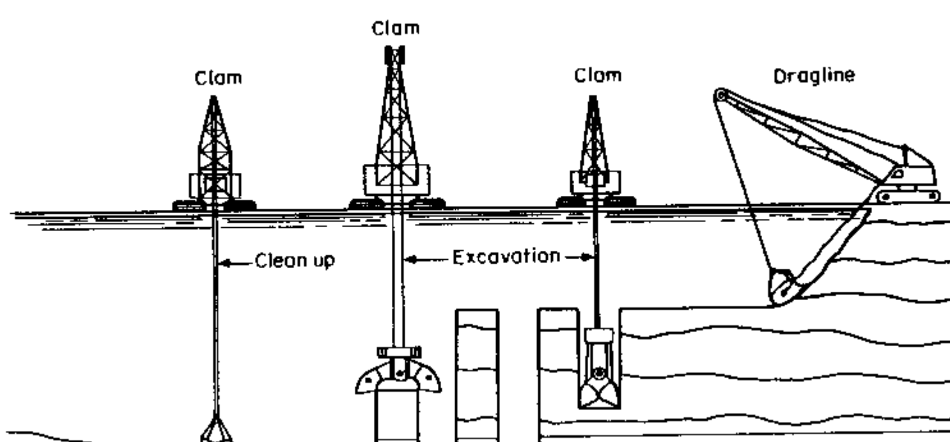


Figure 9.15. Slurry trench excavation by dragline and clamshell (Xanthakos 1979).

Table 9.1. Typical gradation limits for slurry trench backfills (adapted from Xanthakos 1979).

Sieve size	Percentage passing	
	UK and Australian cutoffs	USA cutoffs
75 mm	80-100	80-100
19 mm	40-100	40-100
4.75 mm	30- 70	30- 80
0.6 mm	20- 50	20- 60
0.075 mm	10- 25	10- 30

from the site may be used, although they are unlikely to be suitable where their liquid limit is greater than 60% and may be difficult to mix if they remain in lumps. Table 9.1 gives particle size distributions for the combined backfill material quoted by Xanthakos.

Xanthakos indicates that at least 10% silt and clay fines passing 0.05 mm is needed to give adequately low permeability.

The backfill may be mixed by windrowing, dozing and blading the excavated soil to remove lumps of clay and silt, or pockets of gravel. The soil is then mixed with the bentonite slurry, either by using the same earthmoving equipment, or preferably by using a concrete batch plant type mixer to ensure a uniform product.

The backfill is placed into the trench by gradually pushing it in by bulldozer, such that the backfill slope is between 6H to 1V and 8H to 1V. To start backfilling the initial slope should be formed by lowering the backfill using say a clamshell bucket, since dropping the backfill through the slurry would cause segregation. A gap of 15 to 45 m is left between the excavation face and toe of backfill to ensure proper cleanup of the trench base can be carried out.

No attempt is made to densify or compact the backfill.

Xanthakos (1979) indicates that in some cases 0.6-0.9 m of concrete may be placed at the bottom of the trench to provide protection against erosion of the base under seepage flows. The top of the trench must be protected from drying by placing earthfill over the top. Settlement of the cutoff is quoted by Xanthakos (1979) as being between 25-150 mm for trenches 15 to 25 m deep and 25 m wide. Less settlement is observed in narrower trenches. Settlement mostly takes place in the first 6 months.

9.3.4 *Grout diaphragm wall*

Grout diaphragm walls are excavated continuously in panels as shown in Figure 9.16, with the trench supported by a cement/bentonite slurry. This slurry is left in the trench and cures to give a low strength, low permeability compressible wall. The panels are excavated in the sequence shown in Figure 9.16, with the secondary panels being excavated before the slurry has hardened excessively in the primary panels but has hardened sufficiently to be self supporting. This obviates the need for end support for the panels as is used for cast in place diaphragm walls (see Section 9.3.5)

The trench may be between 0.5 and 1.5 m wide (ICOLD 1985) but will often be nearer the narrow limit for economy.

Excavation is carried out by grab buckets or clamshells. While ICOLD (1985) make no definitive statement on maximum practical depth it can be inferred that grout diaphragm walls may be used to at least 50 m depth.

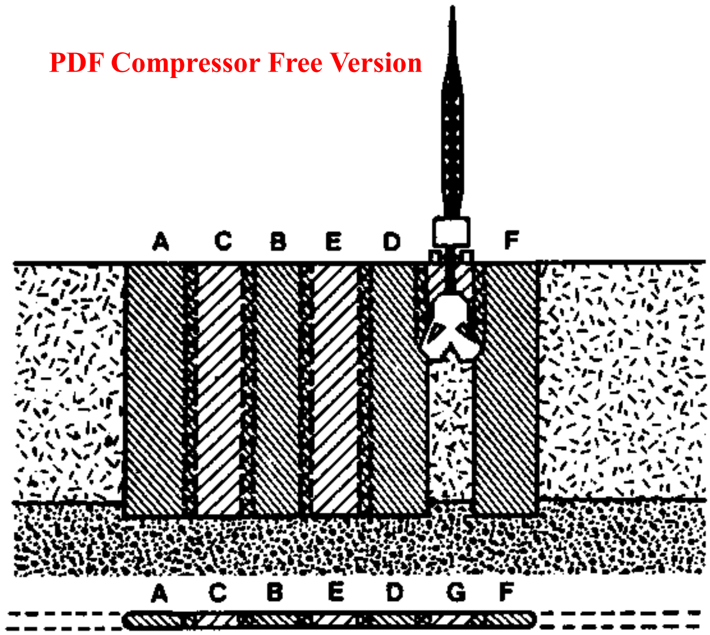


Figure 9.16. Grout diaphragm wall, 1) Order of construction of panels A, B, C, D, E, F, G; 2) ABDF primary panels, CEG secondary panels (ICOLD 1985).

(ICOLD, 1985) indicate that the cement/bentonite grout will usually have the following range of composition per cubic metre of grout:

– 80-350 kg cement

– 30-50 kg bentonite

Xanthakos (1979) indicates that typical mixes will be

15-20% – cement

2-4% – bentonite

5-10% – sand and gravel

The water cement ratio (by weight) will be between

4:1 and 10:1 – for ground granulated blast furnace cement (BLF)

3.3:1 and 5:1 – for portland cement (P)

The blast furnace cement has greater resistance to attack by aggressive groundwater (pore water) which dissolves the free lime in the cement, and selenitic water. These result in formation of tricalcium sulphoaluminates which destroy the hardened grout by expanding. Alternatively fly ash may be added in proportion 10 to 100% by weight of cement (ICOLD 1985).

Retarders are added to the mix to control the curing process mainly to delay the initial set.

The addition of cement, which has free lime ($Ca(OH)_2$ and/or gypsum ($CaSO_4$) to the bentonite slurry makes it flocculate because of cation exchange of the Ca^{++} ions for Na^+ ions in the bentonite. The slurry remains stable (i.e. doesn't bleed) but does not form such an effective filter cake on the sides of the trench as bentonite, and hence losses are greater. Xanthakos (1979) indicates losses may be up to 100% of the trench volume. This increases costs but gives a greater effective wall width. Admixtures can be used to reduce the flocculation effect and the use of

BLF cement (with less free lime) instead of portland cement assists.

The cement bentonite grouts commonly have very low strength compared to concrete. ICOLD (1985) indicate an unconfined compressive strength of 100 kPa at 28 days, 150 kPa at 90 days. The strength is affected by water/cement ratio and cement type. The grout is able to withstand considerable plastic deformation to accommodate settlement due to embankment construction.

9.3.5 *Diaphragm wall using rigid or plastic concrete*

Diaphragm walls are excavated in alternating panels as shown in Figure 9.17 with the panel supported by bentonite. The wall is constructed by tremie pipe placement of concrete or cement-bentonite concrete ('plastic concrete').

The ends of each panel are supported by a steel 'stop-end' tube as shown in Figure 9.18. The pipe is removed after initial set of the concrete leaving a half round key which is used as a guide for the excavating tool, thus reducing potential misalignment of panels and leakage.

It should be noted that for dam cutoffs, the walls are not usually reinforced with steel.

The steel pipe should be the same diameter as the trench so concrete does not leak past the tube. Figure 9.19 shows the effect of overbreak in a gravel layer, allowing concrete to surround the tube and make it difficult to remove the tube.

Other joint systems may be used to achieve a better contact between adjoining panels and improve water tightness. Some examples are given in Figure 9.20

The trench wall thickness is generally 0.6 m for walls up to about 30 m deep increasing to 1 or 1.2 m for deep walls e.g. 50 m. The added width is required to assist in maintaining overlap between adjacent panels. The usual specified tolerance for verticality is 1/100 or 1/200, with some instances of 1:500 being required (Xanthakos 1979).

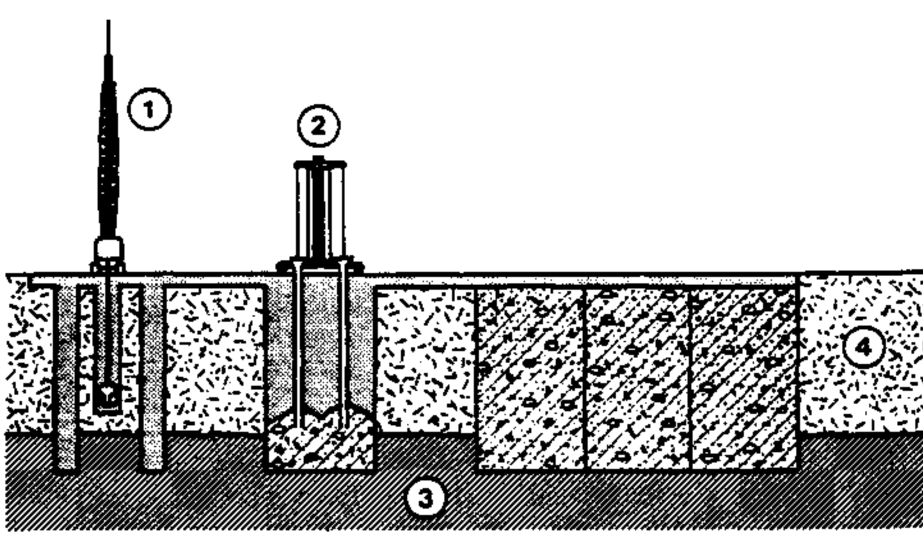


Figure 9.17. Diaphragm wall cutoff, 1) excavation; 2) concreting; 3) substratum; 4) permeable layer (ICOLD 1985).

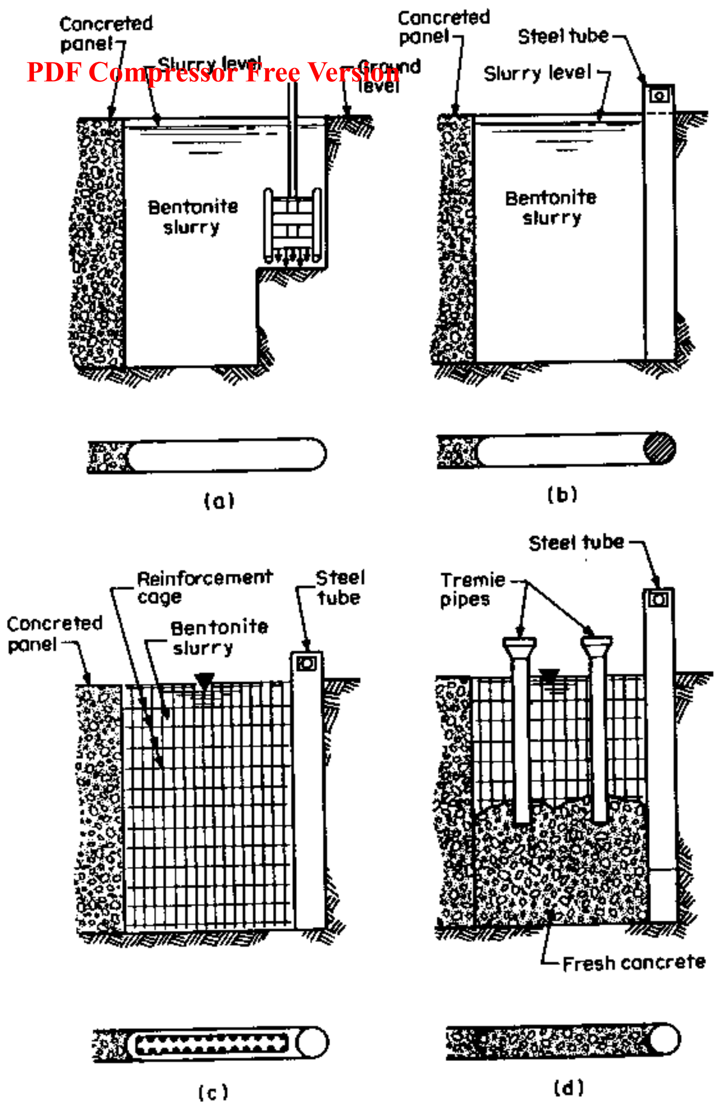


Figure 9.18. Typical construction sequence of a diaphragm wall executed in four stages: a) excavation; b) insertion of steel tubing; c) placement of reinforcement cage; d) concrete placement (Xanthakos 1979).

Excavation is carried out by clamshell, scraping bucket, or rotary drilling equipment. Details are given in Section 9.3.6.

The walls may be constructed of conventional concrete and often are, for building construction work. However, for dam applications, concrete is too rigid. As the soil mass surrounding the wall compresses under the weight of the embankment during its construction, and the water load as the dam is filled, substantial loads will be shed onto the wall by negative skin friction. This can cause crushing of the wall, and penetration of the wall into the dam fill.

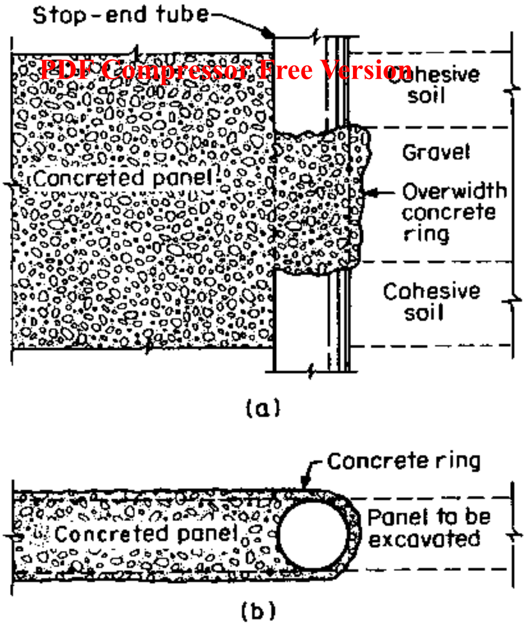


Figure 9.19. Penetration of concrete beyond the stop-end tube due to overbreak a) partial elevation; b) partial section (Xanthakos 1979).

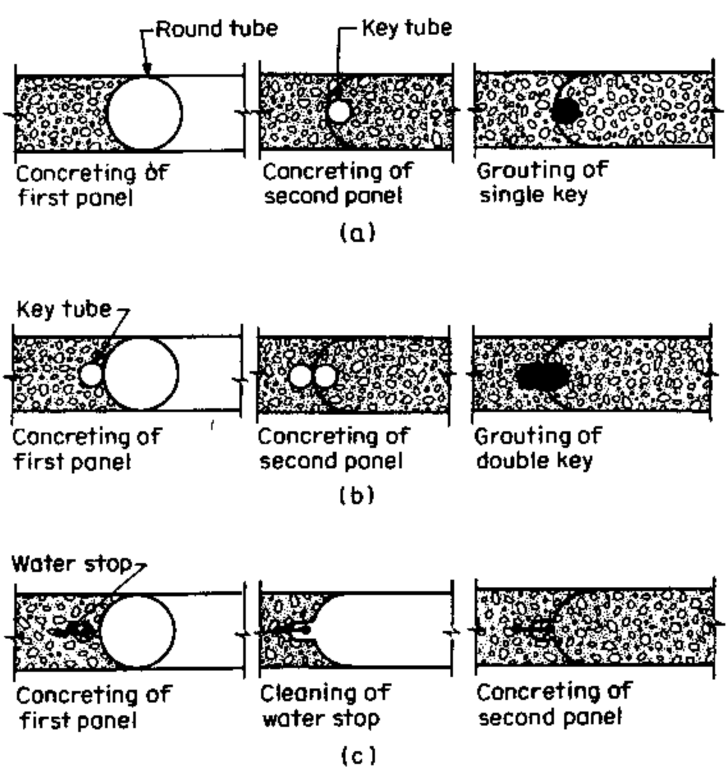


Figure 9.20. a) Single key joint; b) Double key joint; c) Water stop joint (Xanthakos 1979).

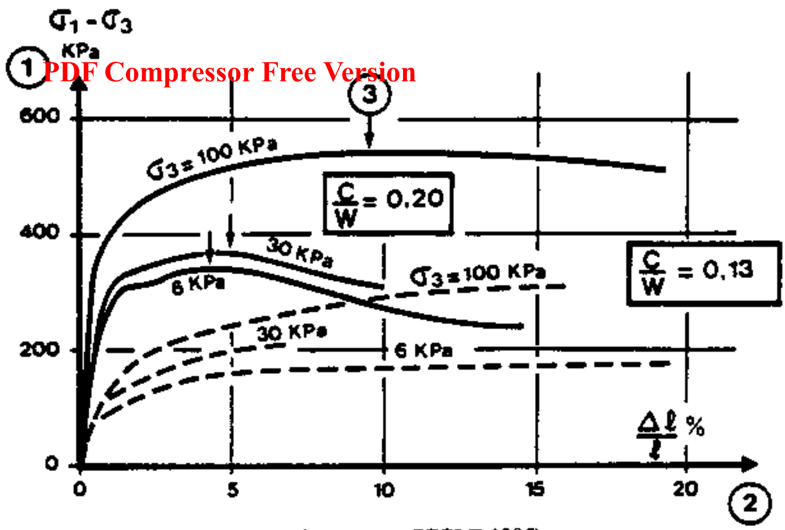


Figure 9.21. Triaxial tests on plastic concretes (ICOLD 1985).

ICOLD (1985) indicate that at Manicougan Dam a 50 m cutoff was successfully constructed with concrete with the following composition:

Portland cement – 200 kg
Water – 100 litres
Sand – 433 kg
19 mm gravel – 452 kg.

For most dam applications it is preferable to use a plastic concrete backfill ie. a concrete with bentonite added. ICOLD (1985) suggest that the following properties are desirable

The Youngs Modulus of the wall should relate to that of the soil, i.e.

– $E_{wall} \leq 5 E_{soil}$.

– Failure strain should be high. Figure 9.21 shows some examples.

– Unconfined strength. A high strength is not important, in the order of 1-2 MPa (Xanthakos suggests an upper limit of 2 MPa).

ICOLD (1985) indicate that the composition of plastic concretes should be (per m^3):

400-500 litres – bentonite slurry
100-200 kg – cement
1300-1500 kg – well graded aggregates less than 30 mm size.

The water cement ratio will be between 3.3 to 1 and 10 to 1 according to the type of cement, the higher values being for BLF cement, the lower for portland cement.

The bentonite slurry is to keep the cement and aggregates in suspension during placement, and assure plasticity and low permeability. The percentage (by weight) of bentonite to water varies from 2 to 12% according to its hydration. The marsh funnel viscosity should be 50 seconds.

If coarse aggregates are replaced by medium to fine sand the composition should be:

375-750 litres – bentonite slurry
75-290 kg – cement
500-100 kg – medium to fine sand.

The concrete (either conventional or plastic) must be placed in the wall by a tremie pipe to avoid segregation. Xanthakos (1979) indicates that it is desirable to complete the concrete pours in less than 4 hours to avoid any significant stiffening of the concrete. For panels up to 3.5-4.5 m long, a single tremie pipe is adequate but for longer panels, two pipes may be needed.

Reinforced concrete guide walls are constructed ahead of the trenching operation, for diaphragm walls filled with grout or concrete. The guide walls

- control the line and grade in the trench,
- support the sides of the trench from heavy construction loads,
- protect the sides of the trench from turbulence and erosion,
- can be braced to support the top of the trench,
- act as a guide trench for the slurry.

Xanthakos (1979) gives details of guide wall design.

9.3.6 *Methods of excavation of diaphragm walls*

Diaphragm walls are usually excavated as a series of panels as shown in Figure 9.22, and each panel as a series of passes as shown in Figure 9.22.

This method can be applied to soft to medium hard, granular and cohesive soils, provided there are not boulders or other obstructions. The minimum 'pass' is about 2 m, the maximum about 6 m.

Excavation will usually be carried out using clamshell type bucket as shown in Figure 9.22. The clamshells are of two types

a) Cable suspended which:

- Are easy to keep vertical.
- Are manouverable.

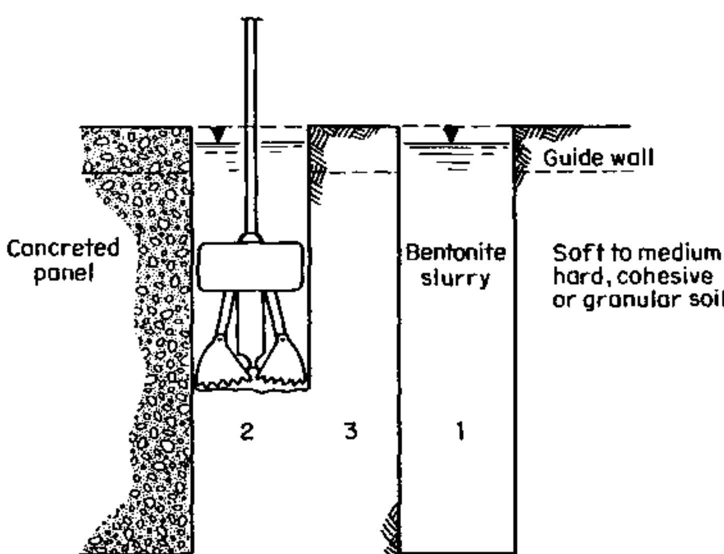


Figure 9.22. Single stage panel excavation with clamshell bucket: Passes 1 and 2 spread of clamshell bucket; Pass 3 spread of clamshell bucket minus clearance for grab to embrace soil (Xanthakos 1979).

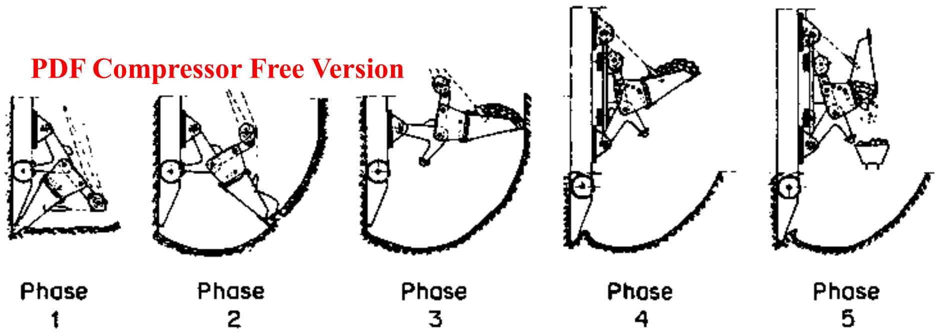


Figure 9.23. Action of a scraping bucket excavator (Xanthakos 1979).

– Can work in limited headroom.

– Depend on their self weight to close the grab. Pulleys or hydraulic systems are used to assist in closing.

– Can be used up to 75 m (cable operated), or 55 m hydraulic.

b) Kelly bar which:

– Are better suited to homogeneous soils.

– Has a maximum depth of 40 m (single piece Kelly) telescoping up to 60 m.

– Requires large headroom.

– Are hydraulically or power operated, with the Kelly bar weight assisting to close the bucket.

Other types of excavation equipment which can be used include:

c) Bucket scraper. This is attached to a Kelly bar as shown in Figure 9.23.

d) Rotary drills. In this technique the excavation is carried out by a drill which has sevei bits mounted on a submersible frame, Figure 9.24 shows one type of such equipment.

The rotary drill bits can excavate into soil or soft rock. The cuttings are suspended in the slurry and flushed from the trench, usually by reverse circulation to improve the lifting capacity. Screens and settling tanks are used to remove the cuttings.

e) Percussion tools. Percussion tools and rock chisels are used where boulders, cobbles or other hard materials are encountered. The broken pieces are removed by clamshell bucket. Figure 9.25 shows examples of this equipment.

In some applications where boulders are present it may be economic to use a two stage operation (Fig. 9.26), where the first stage consists of excavating pilot holes with percussion rotary drill, and the second stage removing the material between the holes with a clamshell bucket. The first stage holes are spaced to leave an intermediate piece 0.3 to 0.6 m narrower than the clamshell bucket grab.

Where bentonite mud is used to support an excavation, as required for virtually all the methods described above, it should be noted that the chemical composition of the groundwater will have an effect on the performance of the bentonite slurry.

In particular, groundwater with a high salt content (i.e. NaCl) or a high calcium ion concentration will cause the bentonite to flocculate. This is caused by contraction of the diffuse double layer around the clay particles as described in Chapter 8.

ICOLD (1985) indicate that:

– fresh water should be used to make the bentonite slurry. Water with up to 5 g/l of salt may

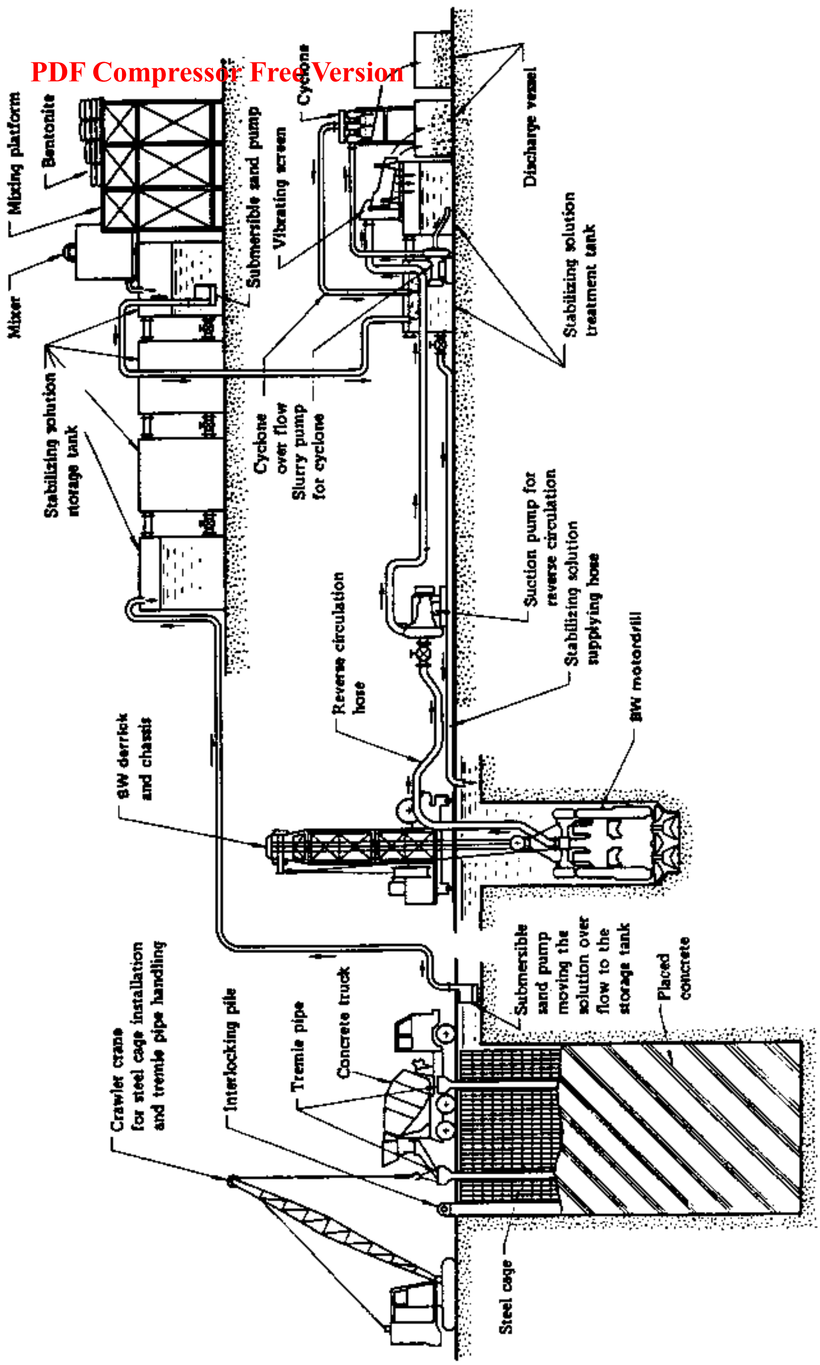


Figure 9.24. Rotary drill technique (Xanthakos 1979).

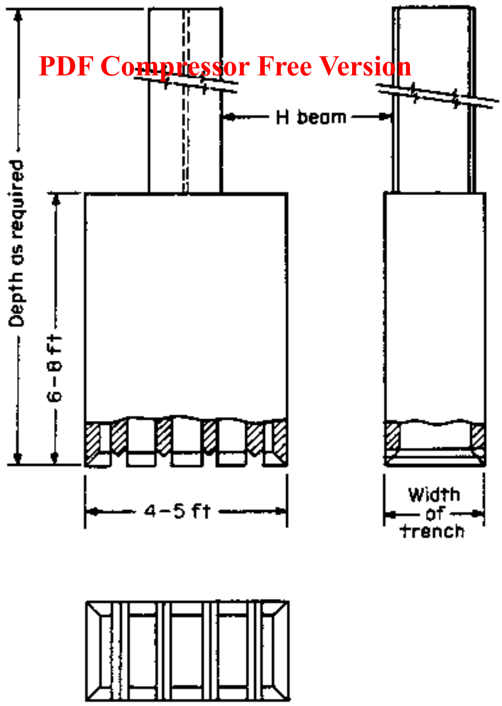


Figure 9.25. Typical chisel details for breaking embedded boulders in site excavations (Xanthakos 1979).

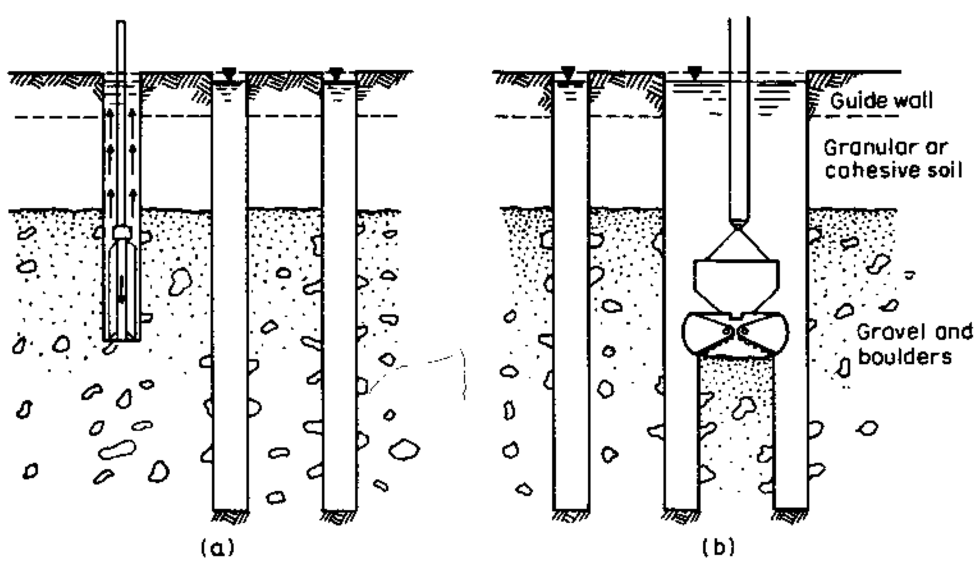


Figure 9.26. Two stage panel excavation with percussive tools and clamshell grab: a) excavation of pilot holes with percussive tools; b) panel excavation with clamshell bucket (Xanthakos 1979).

be used but the bentonite content will have to be increased;

– 'colloid protectors' (called peptizers, or dispersing agents by Xanthakos 1979) may be used to prevent flocculation.

Xanthakos (1979) indicates that even if trenching into soil with groundwater of high salts content, even of seawater quality, the slurry only takes up ionic salt, and the flocculation can be controlled. If only saline water is available, attapulgite may be substituted for bentonite.

More information on muds can be obtained from Xanthakos and from Chilingarian and Vorabutr (1983).

9.3.7 *Permeability of cutoff walls*

ICOLD (1985) indicate that the permeabilities which can be achieved for cutoff walls are:

10^{-7} to 10^{-8} m/sec – for grout walls
10^{-8} to 10^{-9} m/sec – for plastic concrete walls
10^{-9} to 10^{-10} m/sec – for concrete
compared to 10^{-6} m/sec for a grout curtain.

Powell & Morgenstern (1985) reviewed published case histories and concluded that the range of permeabilities achieved was as shown in Figure 9.27.

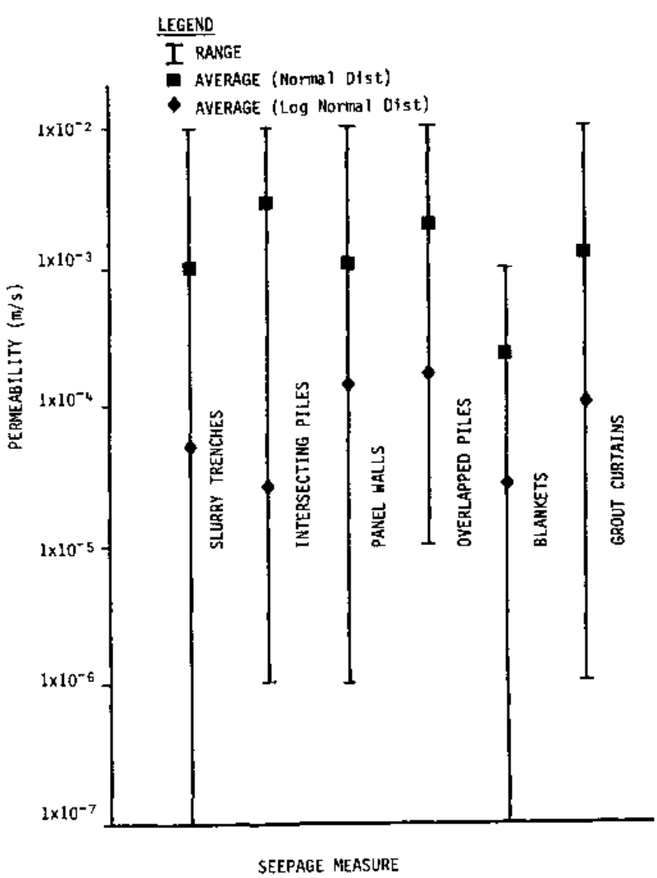


Figure 9.27. Permeability of seepage control measures (Powell & Morgenstern 1985).

It can be seen that there is a considerable difference between the ICOLD and Powell and Morgenstern figures. This is a reflection of the difficulty in maintaining quality control and achieving adequate cutoff between the cutoff wall and the foundation.

The ICOLD (1985) figures are more likely to reflect the properties of the materials in the cutoff wall, while the Powell & Morgenstern (1985) values reflect the equivalent permeability of the constructed wall.

CHAPTER 10

Stability analysis

10.1 GENERAL PRINCIPLES

As discussed below the analysis of the stability of dam embankments should always be carried out using effective stress methods. For such an analysis the critical issues are:

a) The potential mechanism and geometry of sliding.

b) Pore pressures.

c) Shear strength (effective cohesion and effective friction angle).

For dam embankments there are generally three potential failure mechanisms which have to be assessed

– downstream slope for steady state seepage,

– upstream slope for drawdown,

– downstream and upstream slope for the construction condition.

The actual geometry of potential sliding for those cases will be determined by trial and error, but must also allow for zones of lower strength in the embankment, and particularly for lower strength zones in the foundation including:

– fissured soils for an embankment constructed on a soil foundation,

– existing landslide planes,

– bedding plane shears in a rock foundation,

– joint and bedding surfaces in the dam abutment which may combine to make a portion of the embankment potentially unstable,

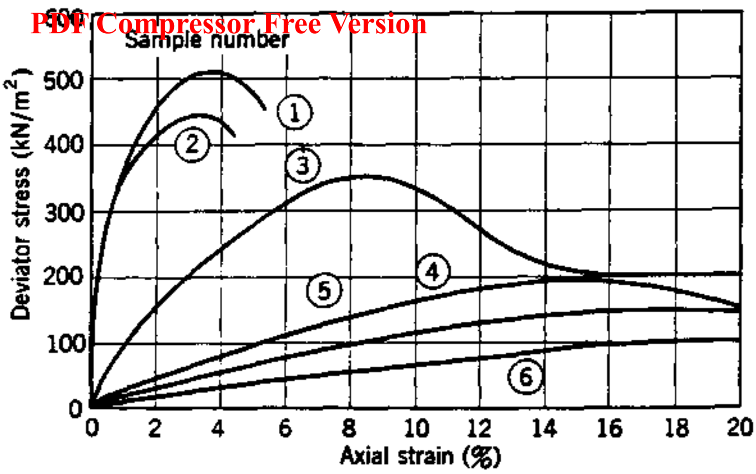
– high pore pressure zones in the foundation, giving a reduction in strength and hence stability.

Most failures which have occurred in dam embankments have been due to not recognising the presence of these lower strength zones, or attributing the wrong strength to them, e.g. peak strength being used when residual was more appropriate, or residual strength being overestimated.

The use of total stress methods of analysis is not recommended because

– The total strength (i.e. undrained shear strength Su) for a material is difficult to determine accurately, with the potential error being much greater than that for effective stress parameters c', ϕ' e.g. for a compacted clay, Su is affected by the degree of compaction and in particular by compaction water content. The strength can be changed by an order of magnitude by these factors, as shown in Figure 10.1.

– It is difficult to monitor and check the undrained strength in the embankment, but it is relatively easy to monitor pore pressures and compare these to those predicted. Since pore



Sample No.	Moulding water content
1	OWC–5%
2	OWC–3%
3	OWC
4	OWC+1%
5	OWC+2%
6	OWC+4%

Figure 10.1. Dependence of undrained strength of compacted clay on compaction density and water content (Lambe & Whitman 1979). Reprinted by permission of John Wiley & Sons Inc.

pressure is the most sensitive variable in effective stress analysis it means that monitoring is relatively straightforward if effective stress methods are being used

Total stress methods have often been used for analysis of the construction condition, see Sherard et al. (1963). Eisenstein & Naylor in ICOLD (1986) show no specific preference for effective stress methods but do indicate the trend is towards their use. The authors' opinion is that the case for effective stress methods is overwhelming even for the construction conditions.

An exception where total stress methods may be applied is the rather unusual case where a dam embankment is to be constructed on soft clay. In this case the short term undrained strength condition is critical and undrained (total stress) analysis is appropriate. Details of analysis for this condition are given in Fell et al. (1987). Total stress methods may also be appropriate under earthquake loading.

Penman (1982) also argues the case for use of undrained strength (at least for construction control) for dam cores which are constructed of very wet clay.

10.2 ESTIMATION OF PORE PRESSURE

10.2.1 *Steady state seepage pore pressures*

Pore pressures for the steady state seepage condition are estimated by calculating the flownet for the embankment section either by graphical techniques or more commonly now by finite element methods. These techniques are described in detail in other references, e.g. Cedergren (1967), Zienkiewicz & Parekh (1970) and Desai (1975) and are not covered here.

The following issues are important when calculating the flownets for embankment dams.

10.2.1.1 *Zoning of the embankment*

Embankment zoning clearly has a vital role in determining the pore pressures in the embankment. This is discussed in Chapter 1.

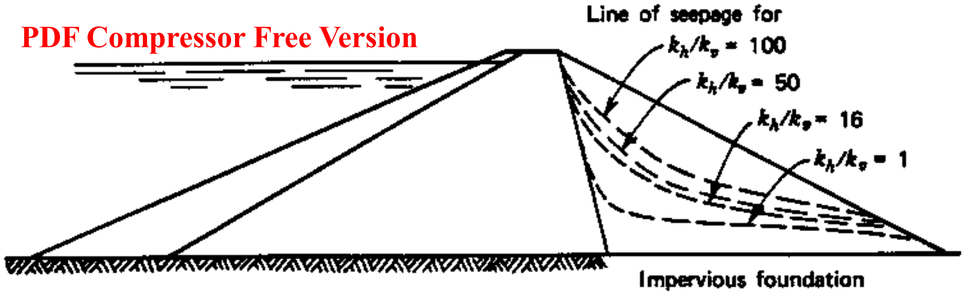


Figure 10.2. Effect of k_H/k_V on pore pressures in a zoned earthfill embankment (Cedergren 1972). Reprinted by permission of John Wiley & Sons Inc.

10.2.1.2 *Anisotropic permeability of embankment earthfill*

Earthfill in dam embankments is compacted in layers, and the action of rolling each layer, possible drying and cracking of the surface of each layer, and greater compaction of the upper part of the layers compared to the lower part will almost invariably lead to the horizontal permeability (k_H) being greater than the vertical permeability (k_V). It would not be unusual for $k_H/k_V \geq 9$ and even as high as $k_H/k_V \geq 100$. This has a marked effect in pore pressures in an embankment, particularly if no seepage control measures such as a vertical drain are incorporated in the design. Figure 10.2 shows pore pressures for a homogeneous dam with horizontal drain for $k_H/k_V = 1, 16, 50$ and 100. It can be seen that pore pressures on the downstream slope are affected greatly by the permeability ratio.

The authors' advice is to design all embankments on the assumption that k_H/k_V is at least 9. For larger more critical dams, internal zoning should control seepage to such a degree that the stability is not greatly sensitive to k_H/k_V. The authors' own practice for earth and rockfill embankment with a vertical drain is to ignore the flownet effect i.e. assume the flowlines are horizontal. This is conservative as it implies $k_H/k_V = \infty$ but only affects the factor of safety marginally over that if $k_H/k_V = 9$ was used and allows one to be confident that pore pressures have been estimated conservatively in some cases, allowing adoption of a lower factor of safety for the design.

10.2.1.3 *Foundation permeability*

The permeability of the foundation (compared to that of the embankment) has an important effect on the seepage flownet. No dam foundation is 'impermeable' and in the majority of cases, even a rock foundation will have a permeability greater than that of compacted earthfill, eg. most rocks have a permeability of between 1 and 100 lugeons (10^{-7} to 10^{-5} m/sec) compared to compacted clay earthfill with a k_V 10^{-7} to 10^{-9} m/sec. Soils in a dam foundation (which will usually be affected by fissures, root holes or layering during deposition) will almost invariably have a permeability far greater than compacted earthfill eg. in the range 10^{-5} to 10^{-6} m/sec. Figure 10.3 shows the effect of such permeability contrasts on the seepage flownet for a homogeneous earthfill embankment (Mardi Dam) and the marked effect on pore pressure.

The following further points are made on Mardi Dam:

1) Figure 10.3a shows the calculated seepage equipotentials for $k_H/k_V = 15$ in the embankment. These give pore pressures compatible with those observed in the embankment.

2) For both cases (a) and (b), most seepage from the reservoir is through the foundation – equipotentials in the embankment upstream of the centreline are nearer horizontal than vertical.

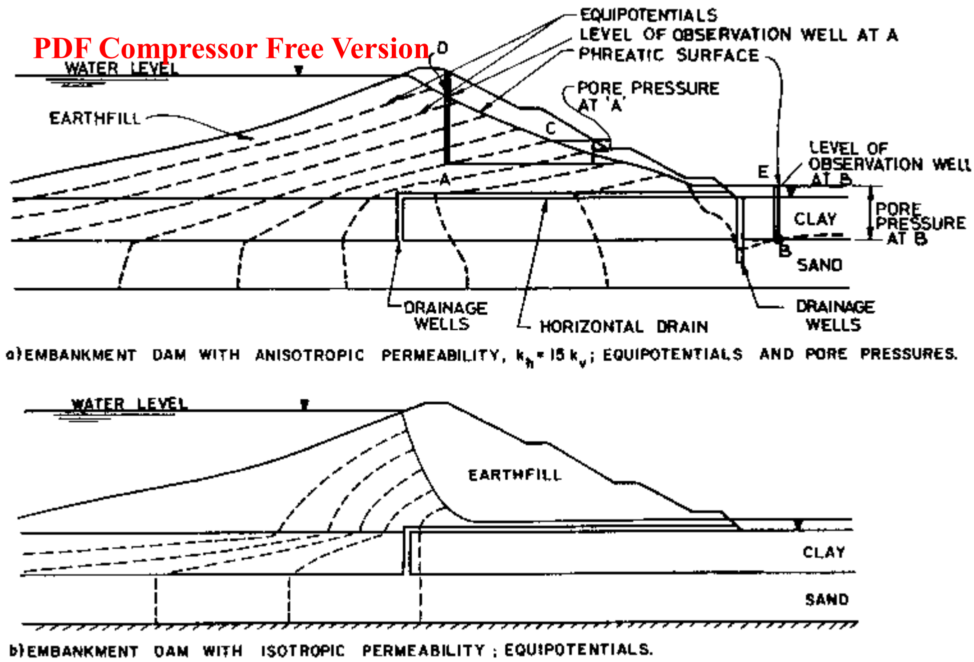


Figure 10.3. Effect of foundation permeability on seepage flownet at Mardi Dam (PWD of NSW 1985, Small 1985 and Walker & Mohen 1987).

3) Even though k_H/k_V for the dam was ≈ 15, and seepage was emerging on the downstream face of the embankment, flow in the downstream part of the dam was towards the horizontal drain, yielding relatively low pore pressures.

4) The upper foundation layer has a lower permeability than the lower layer, and the confining effect led to pore pressures in the lower layer being above ground surface at the downstream toe of the embankment. To remedy this situation pressure relief wells were constructed which fully penetrated the lower (sand) layer. The original wells shown in Figure 10.3 were only partially penetrating.

Figures 10.4 and 10.5 show the effect of foundation permeability on the seepage flownets for Lungga Dam, Solomon Islands, which was to be constructed on permeable alluvial soils.

The analysis for Lungga Dam includes the use of a diaphragm wall which penetrates part way through the highly permeable foundation sands and gravels. Being partially penetrating it only reduces seepage by a small amount (less than 5%) compared to no diaphragm cutoff. Seepage is again largely through the foundation (> 99% of total) and the seepage through the embankment passes directly to the foundation. Figure 10.5a shows the flownet when the upper 3m of alluvium was compacted, resulting in a lower permeability. Figure 10.5b shows the effect of vibroflotation of the foundation which by densification and mixing yields a lower permeability. Lungga Dam was not constructed due to the high overall cost of the project.

It should be noted that for both Mardi and Lungga Dams, the critical failure surface for slope stability passed through the foundation due to the high pore pressures.

Ben Lomond tailings dam is to be constructed on a permeable rock foundation. Figure 10.6 shows the results of seepage analysis.

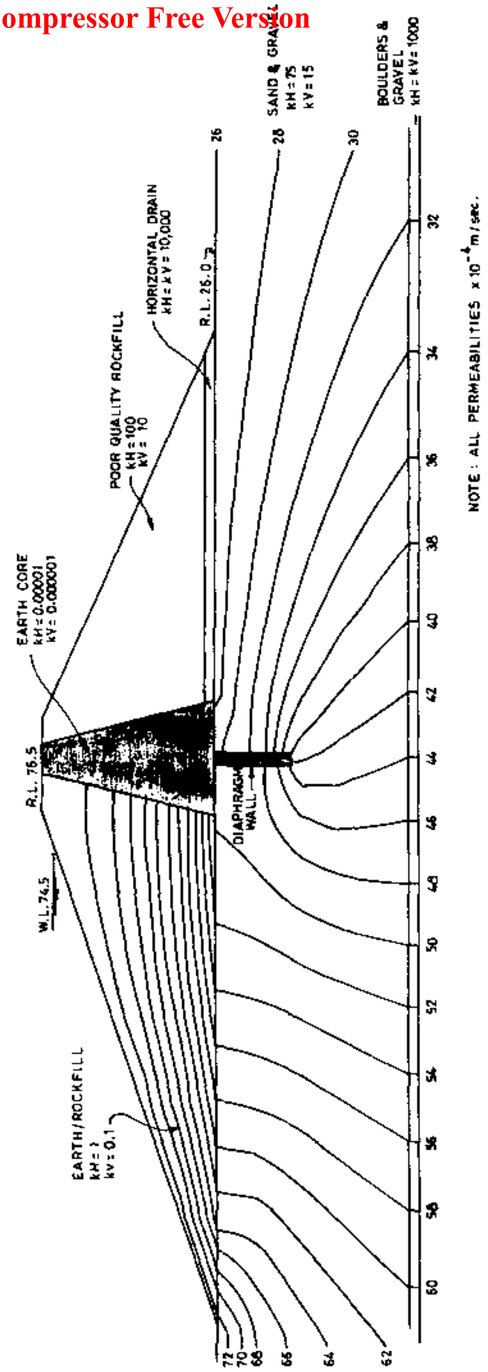


Figure 10.4. Seepage equipotentials Lungga Dam, uniform foundation (Coffey & Partners 1981).

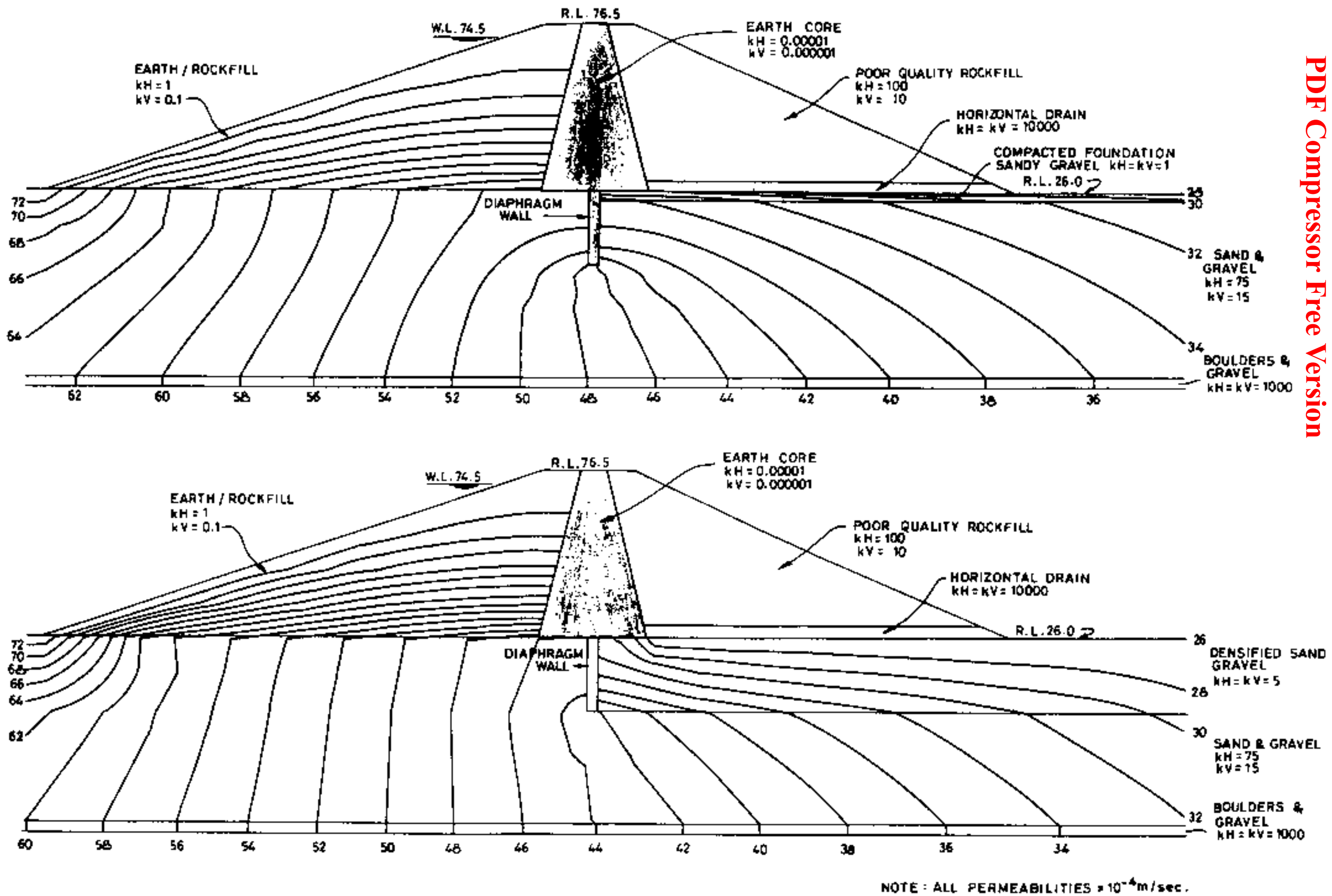
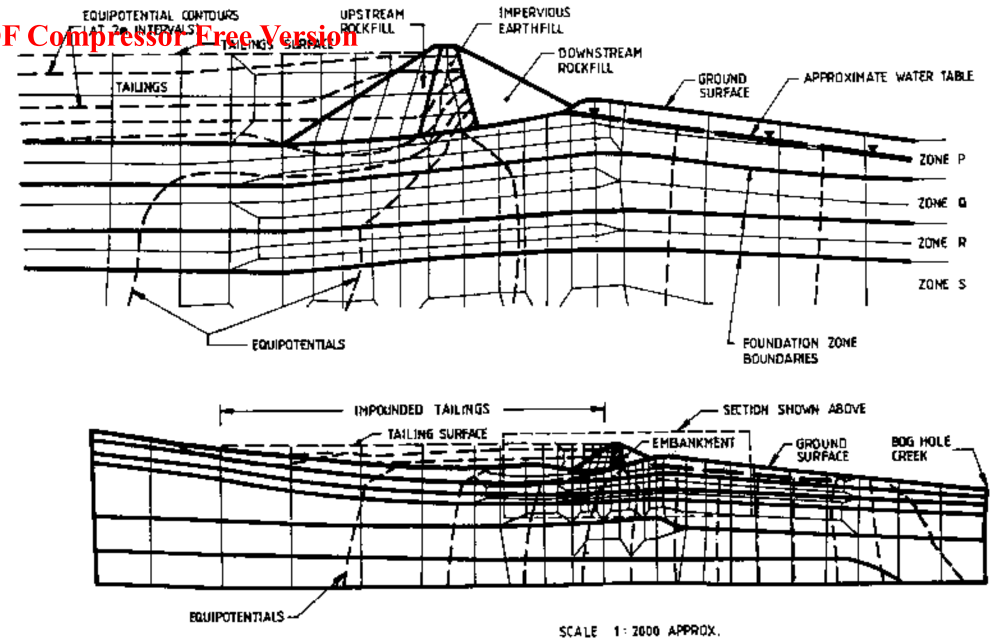
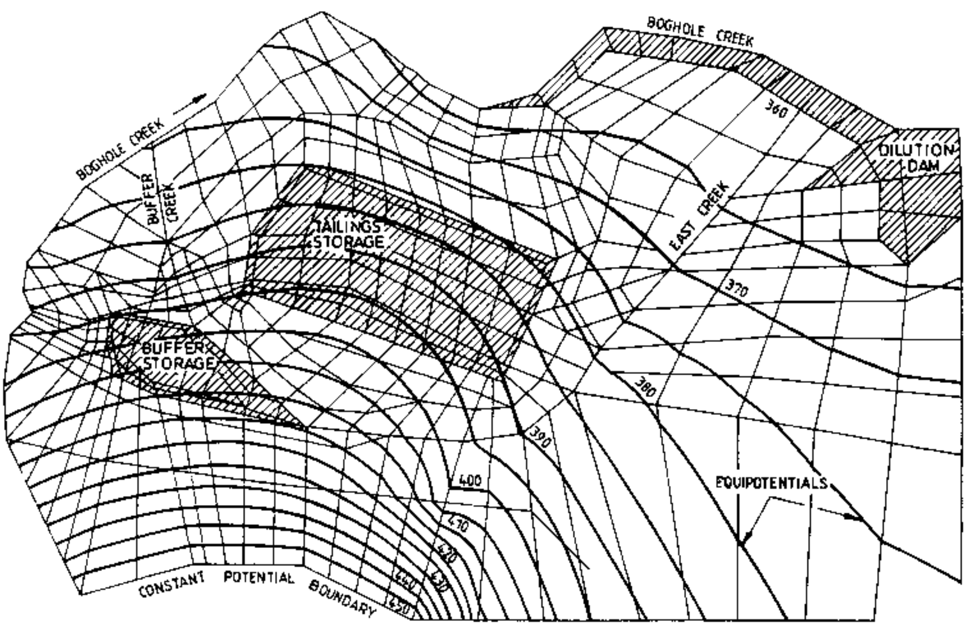


Figure 10.5. Seepage equipotentials Lungga Dam foundation affected by a) upper compacted zone; b) densification by vibroflotation (Coffey & Partners 1981).



(a)

Figure 10.6a. Seepage equipotentials for Ben Lomond tailings dam – sectional models (Coffey & Partners 1982).



(b)

Figure 10.6b. Seepage equipotentials for Ben Lomond tailings dam – plan model (Coffey & Partners 1982).

Seepage from the storage is to be controlled by the relatively low permeability tailings and the upper zone of rock in the foundation, which has clay infilled joints and a lower permeability than the deeper rock

Analysis shows that the seepage will not emerge at the toe of the dam, but in the creek some distance from the dam. This illustrates that when modelling the seepage one must not assume that the phreatic surface is coincident with the ground surface at the dam toe.

Grouting of the rock in the foundation was modelled but resulted in less than 1% reduction in overall seepage and little effect on flownets.

The Mardi, Lungga and Ben Lomond seepage analyses all used finite element methods. Provided the finite element program has good mesh generation and result plotting capability, the analyses can be prepared quickly and economically, allowing checks of sensitivity to different assumptions for permeability. The use of hand methods to draw the flownets for these cases would be time consuming and less accurate.

10.2.2 *Construction pore pressures*

When earthfill is compacted in a dam embankment the water content is usually controlled to be near optimum water content so that compaction is facilitated. The soil will typically be compacted to a degree of saturation around 90 to 95%, will be heavily overconsolidated and have negative pore pressure.

As additional fill is placed, the load imposed on the earthfill lower in the embankment will produce an increase in pore pressure, resulting eventually in positive pore pressures. The pore pressure will depend on the soil type, water content, roller used and magnitude of the applied stress (i.e. height of fill over the layer in question, and lateral restraint).

Larger dams constructed of earthfill with a high water content may develop sufficient pore pressure for this to control the stability of the upstream or downstream slope.

As the pore pressures will build up during construction, and then dissipate with time, the critical condition is that during construction unless the steady state seepage condition is more critical.

10.2.2.1 *Estimation of construction pore pressures by laboratory tests*

Construction pore pressures can be estimated by using Skempton's (1954) pore pressure parameters A and B. In this approach

$$\Delta u = B\,[\Delta\sigma_3 + A\,(\Delta\sigma_1 - \Delta\sigma_3)]$$

where Δu = change in pore pressure

$\Delta\sigma_1$ = change in major principal stress

$\Delta\sigma_3$ = change in minor principal stress.

A and B are pore pressure parameters, which are related to the compressibility of the soil as follows:

$$B = \frac{1}{1 + \dfrac{nC_v}{C_{sk}}}$$

where n = porosity

C_v = compressibility of the voids

C_{sk} = compressibility of the soil skeleton.

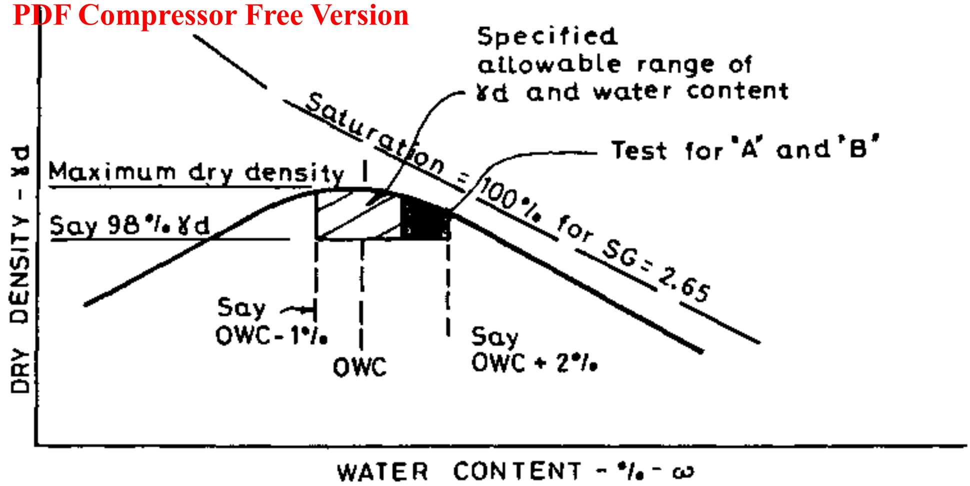


Figure 10.7. Test water content and density ratio for determining A and B parameters.

The pore pressure equation can be rewritten as

$$\frac{\Delta u}{\Delta \sigma_1} = \bar{B} = B\left[\frac{\Delta \sigma_3}{\Delta \sigma_1} + A\left(1 - \frac{\Delta \sigma_3}{\Delta \sigma_1}\right)\right]$$

For saturated soils, B = 1, but as shown in Figure 6.10, B varies considerably with degree of saturation, and the degree of overconsolidation of the soil. 'A' also varies with the degree of overconsolidation of the soil, the stress path followed and with the strain in the soil. Usually A at 'failure' (i.e. at maximum principal stress ratio or maximum deviator stress) is used and is known as A_f. Some typical values are:

Type of clay	A_f
Highly sensitive clays	+0.75 to +1.5
Normally consolidated clays	+0.50 to +1.0
Compacted sandy clays	+0.25 to +0.75
Lightly overconsolidated clays	0 to +0.50
Compacted clay gravels	-0.25 to +0.25
Heavily overconsolidated clays	-0.50 to 0

After Skempton (1954)

The values of A will also be dependent on the lateral strain conditions within the dam embankment. It is normal to assume that K_0 conditions apply ie. that there is zero lateral strain. This is a reasonable (and conservative) assumption for the bulk of the failure surface in an embankment. The use of A at failure is also slightly conservative in most cases.

A more general pore pressure equation was proposed by Henkel (1960) to take into account the effect of the intermediate principal stress. It is

$$\Delta u = B(\Delta\sigma_{oct} + a\,\Delta\tau_{oct})$$

where

$$\sigma_{oct} = \tfrac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$$

$$\tau_{oct} = \tfrac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$$

and a is the Henkel pore pressure parameter. The equivalent Skempton A from Henkel's a parameter for triaxial compression is

$$A = \tfrac{1}{3} + a\frac{\sqrt{2}}{3}$$

For triaxial extension conditions

$$A = \tfrac{2}{3} + a\frac{\sqrt{2}}{3}$$

The pore pressure parameters A and B can be estimated by laboratory tests on the soil to be used in the embankment. 'B' is determined by placing soil compacted to the required water content and density ratio in a triaxial cell, and observing the change in pore pressure Δu for changes in cell pressure under undrained conditions. Under these conditions $\Delta\sigma_1 = \Delta\sigma_3$, and the pore pressure equation becomes

$$\Delta u = B\,[\Delta\sigma_3 + A\,(0)] = B\,\Delta\sigma_3$$

This relationship will not be linear and must be determined over a range of cell pressures. B will be larger for higher cell pressures than for low cell pressures.

To determine A, the soil is placed in the triaxial cell under undrained conditions and sheared. Knowing B from the earlier testing and observing Δu, $\Delta\sigma_1$ and $\Delta\sigma_3$, A can be determined from the pore pressure equation. As explained above, in most cases this will be done under K_o conditions. Head (1985) shows a suitable test set up and lateral strain indicators needed to control the test using conventional triaxial equipment. More accurate estimation of the pore pressure parameters can be achieved by more closely following the stress history of the soil in the embankment using a Bishop-Wesley triaxial cell.

10.2.2.2 *Application of the laboratory test parameters*

To apply this method one must determine:

– What the stresses in the embankment will be.

– The pore pressure parameters A and B relevant to the water content and density ratio at which the soil is placed and the stress conditions in the embankment.

To be conservative in the estimation of pore pressures, the laboratory tests to estimate A and B can be carried out at the upper limit of specified water content (e.g. optimum +2%) and/or degree of compaction (density ratio 98 to 100%, standard compaction). However it must be remembered that at optimum +2% water content, the maximum achievable density ratio may be 97 to 99%, while at optimum water content a density ratio of 100 to 101% may be achievable, and the laboratory testing should account for this (see Fig. 10.7).

The stresses in the embankment due to construction of the dam can be estimated by finite element techniques. ICOLD (1986a) gives a description of the methods and their limitations. It is concluded that:

– A two dimensional model can be used to estimate vertical stresses in a homogeneous dam on a rigid (rock) foundation.

– A three dimensional model which is modelled as 3 to 10 layers progressively being loaded, is necessary to model cross valley stresses in an embankment on a compressible foundation

– The initial stresses locked into the fill during compaction should be incorporated into the model.

In a practical sense it is difficult to carry out 3 dimensional modelling, and the rolling stresses can only be crudely modelled (Naylor 1975, ICOLD 1986). Given the inaccuracuries in predicting pore pressure parameters A and B with any degree of accuracy, it is not worthwhile spending too much effort on modelling the stresses. Two dimensional finite element models along the dam axis, as well as up and down river might give some appreciation of three dimensional problems for narrow or irregular shaped valleys, and compressible foundations.

It should be recognised that it is not practicable to estimate construction pore pressures accurately and that the best one can do is to get an indication of the likely pressures, and how they will build up as the embankment is constructed. It is necessary to monitor pore pressures with piezometers if it is considered they may be critical. Measures can be taken to reduce pore pressures, e.g. by using a lower water content. This was done at Dartmouth Dam (Maver et al. 1978). However by reducing the compaction water content the earthfill will be more brittle and more susceptible to cracking.

The discussion above ignores the effects of dissipation of pore pressure with time in the embankment. Eisenstein and Naylor in ICOLD (1986) discuss how this can be modelled by finite element methods. In most cases the earthfill will have a low permeability and coefficient of consolidation, and drainage paths will be long which will result in long times for pore pressure dissipation. As the estimation of pore pressures is approximate in any case, it would not be warranted to take account of pore pressure dissipation in most cases.

10.2.2.3 *Estimation of construction pore pressures from drained and undrained strengths*

McKenna (1984) cites an approximate method for estimating construction pore pressures developed by P.R. Vaughan. This method was used at Yonki Dam in Papua New Guinea by the Snowy Mountains Engineering Corporation.

The method is based on the effective strength and total strength envelopes obtained from tests on the embankment soil. These are plotted as p-q plots yielding envelopes where for the effective strengths

$$q = p' \tan\alpha' + a'$$

and for total strengths

$$q = p \tan\alpha + a$$

where $\tan\alpha = \sin\phi_u$
$a = S_u \cos\phi_u$
$\tan\alpha' = \sin\phi'$
$a' = c'\cos\phi'$

The pore pressure at failure is given by

$$u_f = \frac{a\,(1 + \cot\alpha') - a'\cot\alpha'\,(1 + \tan\alpha)}{(1 - \cot\alpha'\tan\alpha)} + \frac{1 - \cot\alpha'\tan\alpha}{1 + \tan\alpha}\,\sigma_1$$

$$= \text{threshold pressure} + (\text{slope of curve})\,\sigma_1$$

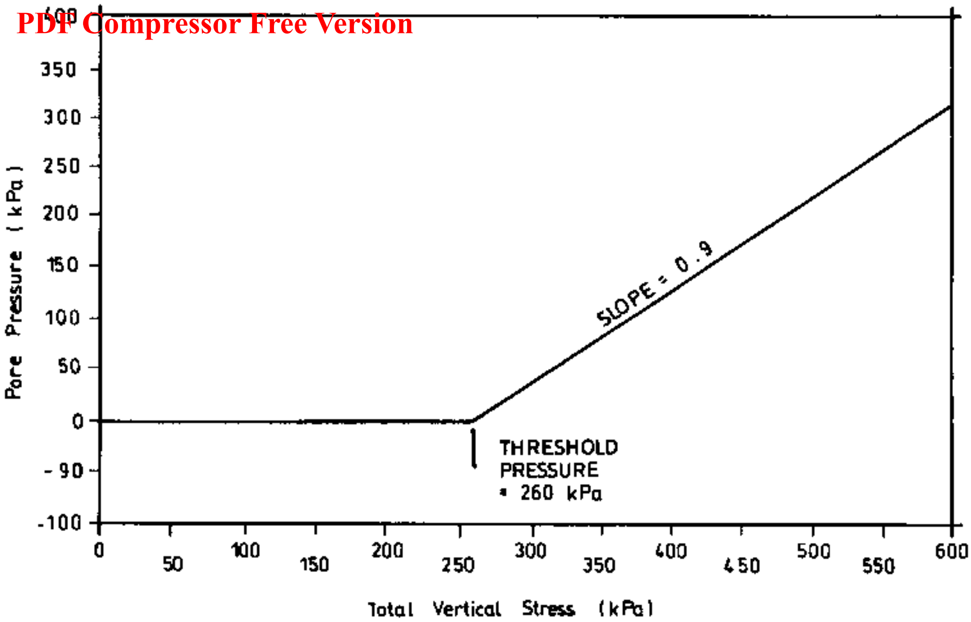


Figure 10.8. Predicted construction pore pressures vs total vertical stress for Yonki Dam using Vaughan method.

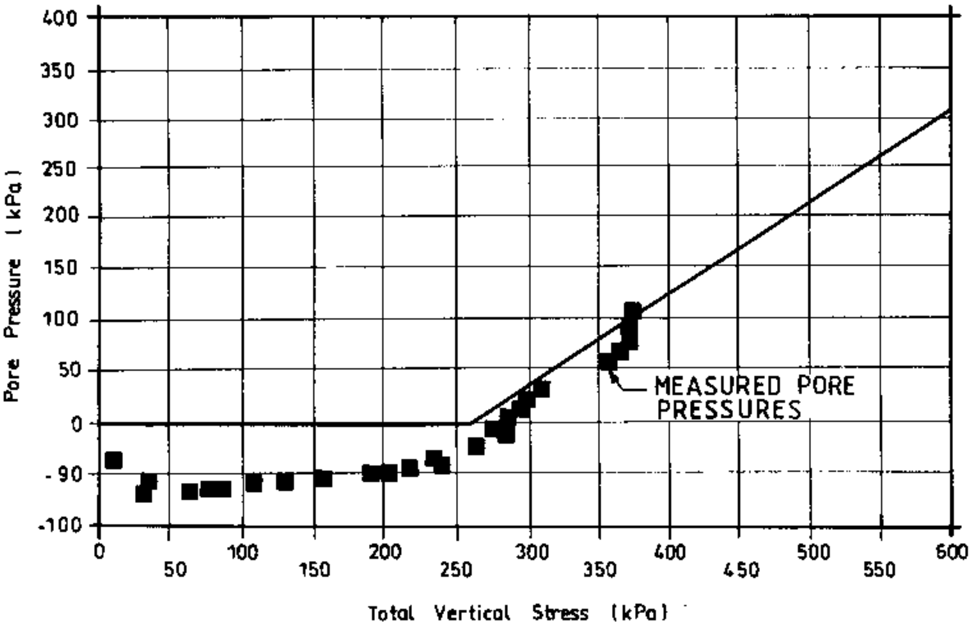


Figure 10.9. Observed construction pore pressures for Yonki Dam – courtesy of the Snowy Mountains Engineering Corporation.

In this σ_1 is the total vertical stress in excess of the threshold pressure.

e.g. for $c'=0$, $\phi'=30°$, and $\phi_u=2°$, slope of curve = 0.9, 'x' intercept = 3.23 Su.

Hence if the dam is constructed so that Su = 80 kPa, the predicted pore pressures would be as shown in Figure 10.8.

In practice there are negative pore pressures in the embankment below the threshold pressure and the slope of the curve increases with increasing vertical stress.

Figure 10.9 shows results from Yonki Dam.

10.2.3 *Drawdown pore pressures*

When the level of a reservoir behind an embankment is lowered, the seepage flownet under the reservoir full condition is altered. The extent to which pore pressures will affect the stability of the upstream slope depends on:

– the change in water level,
– the rate of change,
– embankment zoning and geometry,
– relative permeability of the embankment and the foundation.

Some of these points are illustrated in Figure 10.10 and discussed below.

1) In most cases the assumption is made that drawdown is 'instantaneous,' and hence the flownet changes from the steady state case to the drawdown case instantaneously. This is somewhat conservative but is accounted for in limit equilibrium analyses by generally accepting a lower factor of safety than for the steady state seepage case.

2) The pore pressures in the embankment are immediately reduced on drawdown due to the unloading effect of removing the water. Hence pore pressures on the surface of earthfill in Figures 10.10a to d are all zero.

3) In Figure 10.10a the foundation permeability is low compared to that of the earthfill, and flow will be towards the upstream face. This gives relatively high pore pressures compared to Figure 10.10b. In the latter case, the foundation is more permeable than the earthfill, so flow lines will be near vertical, and equipotentials near horizontal, giving pore pressures close to zero.

4) In Figure 10.10c, horizontal drains are incorporated in the upstream slope. These cause flow lines to be near vertical on drawdown (except near the low permeability foundation) and result in low pore pressures.

5) In Figure 10.10d, the flow lines are influenced by the presence of the filter zones and rockfill, giving lower pore pressures than for Figure 10.10a at least in the upper part of the slope.

Drawdown flownets and pore pressures can be obtained by graphical methods, or finite element analysis. Eisenstein and Naylor give details of the latter in ICOLD (1986). Factors which must be considered in the analyses are:

– permeability anisotropy in the earthfill, i.e. allowing for $k_H > k_V$;

– the relative permeability of the foundation and the embankment zones. In practice this means that the foundation should be modelled with the embankment;

– the effect of removal of buoyancy from the upstream rockfill zone (if present) should be allowed for. This will induce increased pore pressures.

The drawdown condition is the controlling case for design of the upstream slope of embankments such as those shown in Figure 10.10 and the analysis is sensitive to the assumptions made. For a central core earth and rockfill dam, the analysis may control the slope, but will not be so sensitive to assumptions.

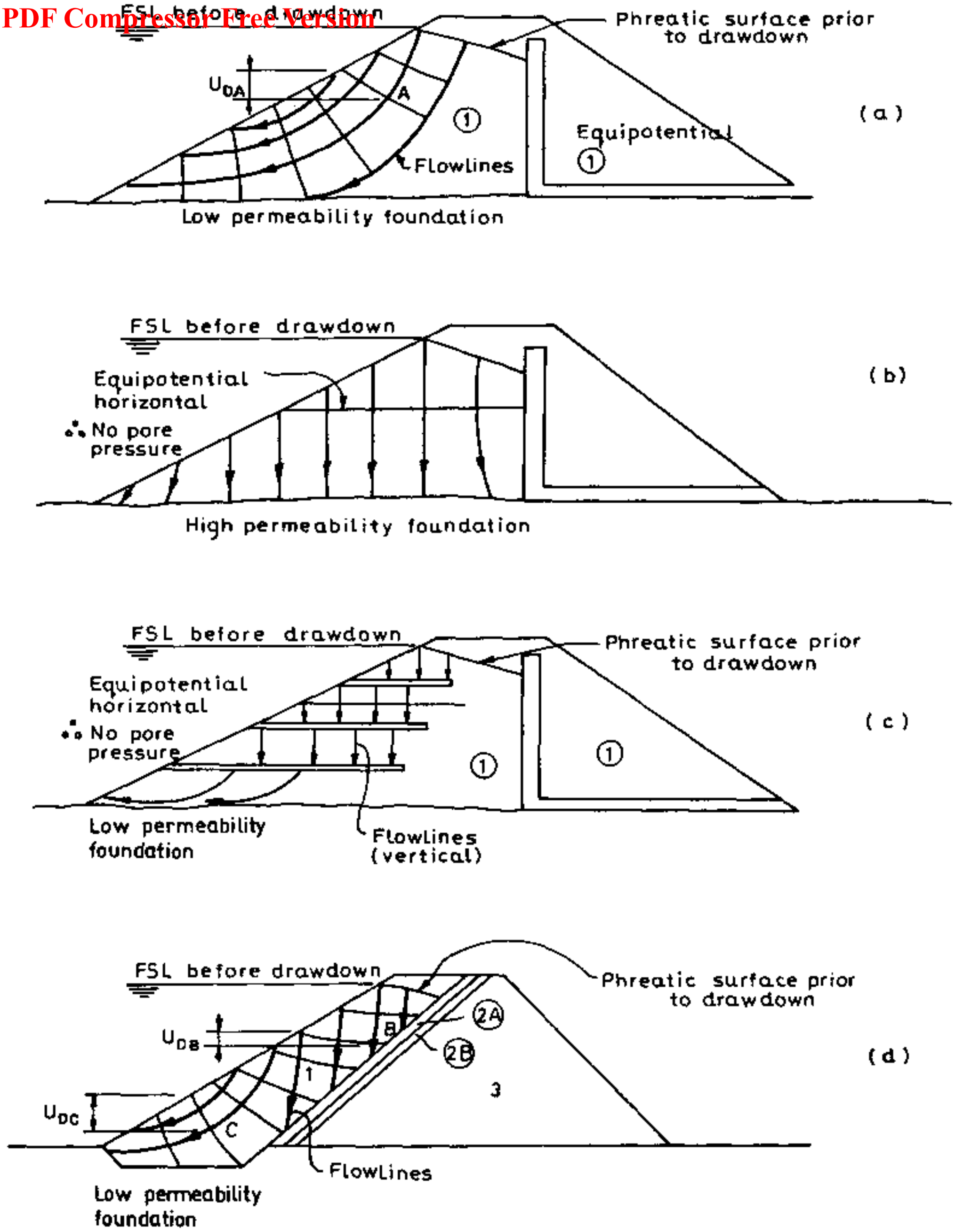


Figure 10.10. Drawdown flownets a) earthfill with chimney drain, low permeability foundation; b) earthfill with chimney drain, high permeability foundation; c) earthfill with horizontal drainage blankets; d) sloping upstream core, low permeability foundation.

10.3 ANALYSIS OF STABILITY

10.3.1 *Methods of stability analysis*

The analysis of stability of dams is almost always carried out using limit equilibrium methods. These methods are well established and described extensively in text books and other literature so will not be presented here. The papers by Mostyn & Small (1987), Graham (1984), Fredlund (1984), Fredlund & Krahn (1977) and Whitman & Bailey (1967) present detailed discussion and comparisons of the methods.

In some cases it is necessary to analyse dams using finite element methods. This may be necessary where stability is marginal, or it is necessary to predict deformations and/or pore pressures. The finite element method cannot alone be used for design because it does not allow quantification of the 'factor of safety.'

Eisenstein and Naylor in ICOLD (1986) give an overview of finite element methods.

Donald & Giam (1988) and Giam & Donald (1988) describe the use of finite element techniques for analysis of the stability of slopes. These describe the use of displacements as a guide to incipient failure is used to allow calculation of a 'factor of safety,' and the use of stress distributions to better predict the critical failure surface.

Finite element methods incorporating strain softening have been developed by Dounias et al. (1988) to analyse progressive failure. The application to Carsington Dam is described in Vaughan et al. (1989) and Potts et al. (1990). They advocate the use of such analysis where high clay content soils subject to strain softening are used for dam construction.

10.3.2 *Analysis of stability of embankment dams*

The following points are made specifically in reference to use of limit equilibrium methods to analyse the stability of embankment dams.

a) The Bishop simplified, Spencer, Fredlund and Krahn, Janbu, Morgenstern Price and Sarma methods will all give similar results for analysis of stability and are acceptable methods. The so called 'standard,' or 'Fellenius' method underestimates the factor of safety and should not be used.

b) In many applications to analysis of stability of dam embankments non circular failure surfaces must be considered. Examples where this is so are given in Figure 10.11.

c) There are no 'rules' for acceptable factor of safety. ANCOLD (1969) recommend:

Downstream slope, steady state seepage: $F \geq 1.5$

Upstream slope, instantaneous drawdown: $F \geq 1.25$ to 1.3

Upstream and downstream slope, construction conditions: $F \geq 1.3$

ANCOLD (1969) suggest these factors of safety should be achieved for conservative assumptions for shear strength and pore pressures and using the 'standard' method of slices.

The authors agree with these recommendations except that the Bishop simplified method (or equivalent) should be used. However, it is essential that analysis be carried out to assess the sensitivity of the factor of safety to assumptions on shear strength, pore pressures and geometry of sliding, and that the embankment is designed to be stable within the widest range of assumptions.

It is recommended also that the analysis be checked using residual strengths, and the highest possible pore pressures, with a factor of safety of at least 1.1 being required. This is particularly important where large differential strains may be encountered, or the dam may be damaged by earthquake.

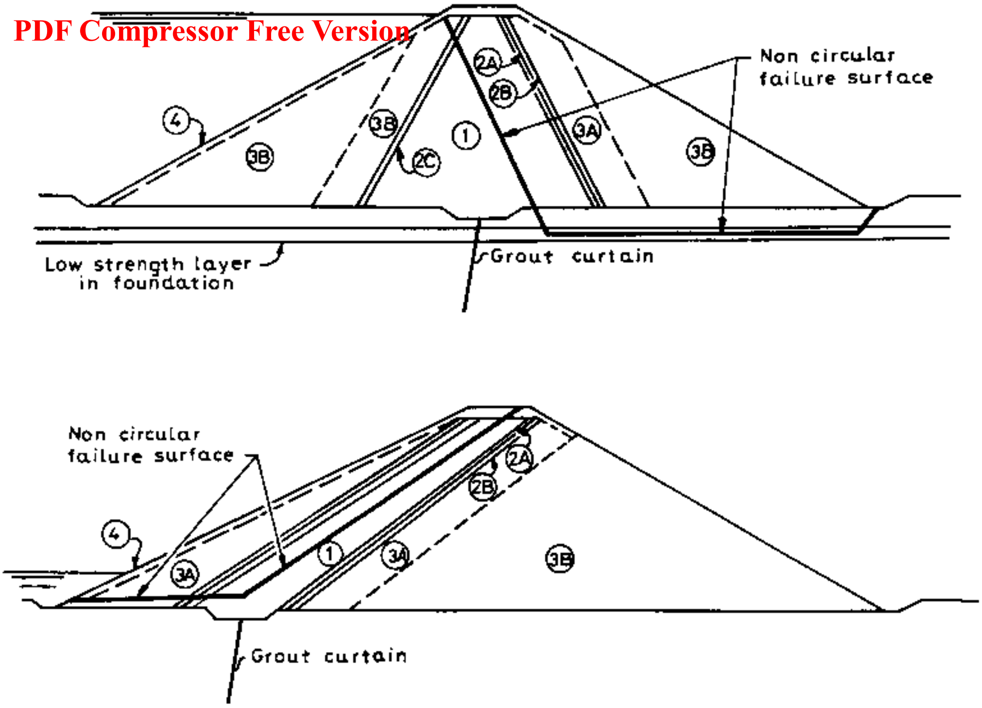


Figure 10.11. Examples where non circular envelopes must be considered.

Table 10.1. US Corp of Engineers loading conditions, required factors of safety, and shear strength for evaluations for embankment dams (National Research Council 1983).

Case	Loading condition	Required factor of safety*	Shear strength for evaluation**
1	Steady seepage at high pool level	1.5	S strength
2	Rapid drawdown from pool level	1.2	Minimum composite of R and S
3	Earthquake reservoir at high pool for downstream slope reservoir to intermediate pool for upstream slope	1.10	R tests with cyclic loading during shear

*Ratio of available strength to shear stress, required for stable equilibrium.

**Terminology from US Army Corps Engineers, R = total stress shear strength from consolidated undrained shear tests; S = effective stress shear strength from drained or consolidated undrained shear tests.

National Research Council (1983) quote Table 10.1 as the US Corp of Engineer's requirements. These are broadly consistent with the requirements discussed above.

d) Critical failure surfaces i.e. those which give the lowest factor of safety, should always pass through the earthfill zone and be likely to cause significant damage if sliding occurred. Failure surfaces which just touch the surface of rockfill (see Fig. 10.12), are often (correctly) designated as giving the minimum factor of safety by computer analyses using automatic search routines, but are not critical to stability because there are no conditions (other than

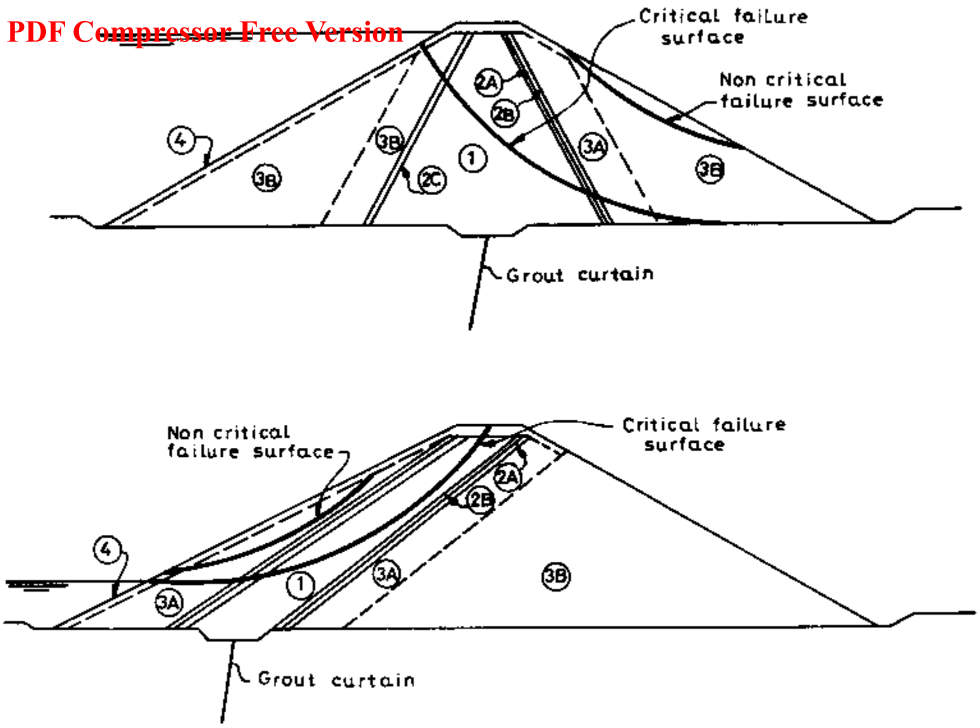


Figure 10.12. Non critical and critical failure surfaces.

earthquake) which will lead to failure of the embankment by this mechanism.

e) Situations which lead to overestimation of factor of safety in embankment dams can usually be related to the assumptions made regarding shear strength and pore pressures, not to problems in the analysis itself, e.g:

– non recognition of bedding plane shears or landslip surfaces in the foundation,
– non recognition of fissuring in the soil foundation,
– high k_H/k_V in embankment fill,
– 'liquefaction' under earthquake loading, leading to loss of shear strength.

CHAPTER 11

Foundation preparation and cleanup

11.1 GENERAL REQUIREMENTS

The degree of foundation preparation which is necessary for a dam embankment depends on:

– the type of dam,

– the height of dam and hazard rating,

– topography of the dam site,

– erodibility, strength, permeability, compressibility of the soil or rock in the dam foundation,

– groundwater,

– climate.

Foundation preparation for the 'general foundation,' i.e. the foundation beneath the bulk of the embankment is quite different from that for the cutoff foundation, i.e. the foundation under the earthfill core of an earth and rockfill dam, or the plinth of a concrete face rockfill dam.

The objectives are:

– General foundation: To remove low strength and compressible material so as to provide a foundation of adequate strength and compressibility to support the embankment. In most cases the general foundation will be of higher strength than the embankment, and will not dictate the embankment stability. In most cases permeability will not be a critical factor for the general foundation.

– Cutoff foundation: To remove highly permeable and erodible material below the general foundation level so as to provide a low permeability non erodible foundation. In many cases, e.g. in soil foundations or rock foundations which are permeable to great depth, a low permeability non erodible cutoff foundation cannot be economically achieved, and other design measures are required. These are discussed in Chapter 9.

In all cases it is important to define the requirements in geological and geotechnical terms which can be identified in the field during construction. This is discussed in Section 11.5.

It is likely that in most cases excavation will proceed in stages, with sufficient clean up to identify whether the geological and geotechnical requirements have been achieved.

11.2 GENERAL FOUNDATION PREPARATION

The requirements for preparation of general foundation are as described below.

11.2.1 *General foundation under earthfill*

11.2.1.1 *Rock foundation*

– Remove topsoil, and weak compressible soil. In most cases this will involve removal of colluvial soil and rock, including boulders to expose an *in situ* rock foundation which may be extremely weathered, variably weathered or in some cases fresh (unweathered). An adequate foundation can usually be identified by near blade refusal of a small bulldozer or excavator.

– Where there are unfavourably oriented weak seams in the rock, e.g. bedding plane shears, or landslide slip planes, these will influence stability and may need to be removed, or the design modified to accommodate them.

– Slope modification may be necessary as described in Section 11.6.

– The surface should be cleaned of loose soil and rock prior to placing earthfill, e.g. with a grader, backhoe or excavator. Intensive cleanup is not generally required. It may be desirable to moisten the surface prior to placing earthfill to maintain adequate moisture in the earthfill for compaction.

11.2.1.2 *Soil foundation*

– Remove topsoil and weak compressible soil consistent with the assumptions made for stability and settlement analysis and design.

– Where soils are fissured, or have landslide slip planes present, these will influence stability and may need to be removed, or the design modified to accommodate them.

– Slope modification may be necessary as described in Section 11.6.

– The surface may be proof rolled with a steel drum or tamping foot roller to assist in locating weak and compressible soil. Rolling to a specified compaction requirement should not be necessary.

– The surface should be cleaned of loose soil and rock prior to placing earthfill, e.g. with a grader, backhoe or excavator. Intensive cleanup is not generally required. It may be desirable to scarify and moisten the surface prior to placing the first layer of soil to assist in 'bonding' the embankment to the foundation.

11.2.2 *General foundation under rockfill*

Rockfill will generally be underlain by a rock foundation (or by a horizontal filter drain, in which case refer to Section 11.2.3):

– Remove topsoil, soil and weathered rock which has a strength lower than the rockfill. In most cases this will involve exposing highly or moderately weathered rock. An adequate foundation may be identified by blade refusal of a bulldozer but for high dams light ripping may be necessary

– Where there are unfavourably oriented weak seams in the rock, e.g. bedding plane shears or landslide slip planes, these will influence stability and may need to be removed or the design modified to accommodate them.

– It is unlikely that slope modification will be required under rockfill other than perhaps overhanging cliffs being removed.

– The surface should be cleaned of loose soil and rock with a bulldozer, grader, backhoe or excavator sufficient to ensure the rockfill is supported on the rock foundation. Intensive cleanup is not required and if there is loosened rock of good quality this may be left in place.

– If there are wide, erodible seams in the rock, oriented such that they are likely to carry seepage water from the storage, it may be necessary to cover them with slush concrete, gunite, or a filter layer. Wallace & Hilton (1972) describe backfilling compressible seams greater than 0.3 m wide with Zone 2B filter material to a depth of up to 0.9 m. In most cases this will not be necessary unless the seams are wide enough to affect stability or pose an erosion risk.

11.2.3 *General foundation under horizontal filter drains*

Horizontal filter drains will generally only be required if the foundation is soil, or erodible rock:

– Remove topsoil and weak compressible soil consistent with the assumptions made for stability, settlement and particle size used for design of the filter.

– Where soil or rock in the foundation is fissured or has landslide slip planes or unfavourably oriented bedding plane shears present, these will influence stability and may need to be removed, or the design modified to accommodate them.

– Slope modification should not be required except to remove overhangs. However, if earthfill is to be placed on top of the filter drain, slope modifications may be needed as described in Section 11.6.

– The surface should not be rolled prior to placing the filter. Rolling will destroy the soil structure and reduce the permeability, making it more difficult for seepage water to flow into the filter drains. It is desirable for the foundation seepage to flow into the filter drain so erosion is controlled, rather than being forced to emerge downstream of the toe of the embankment in an uncontrolled manner.

– On low strength rock, trafficking with earthmoving equipment will continuously break up the surface, necessitating final cleanup with an excavator or backhoe working away from the cleaned up area. This is particularly a problem with weathered schists, phyllites and similarly fissile rocks which break down very easily under equipment.

– Immediately before placing the filter material, the surface should be cleaned of loose, dry and wet soil and rock. This may necessitate intensive work using light equipment and hand methods. Final cleanup should involve an air or air water jet to 'blow' away loose material. In many cases an air-water jet will be too severe and cause erosion.

11.3 CUTOFF FOUNDATION

The requirements for excavation below general foundation level to achieve a suitable cutoff are described below.

It should be noted that each case will be determined on its merits and often there will be trade-offs between the following:

– the desire to achieve a low permeability foundation,

– the depth and hence volume and cost of excavation required to achieve a cutoff,

– the extent of grouting planned in the foundation below the cutoff,

– the protection downstream to control foundation erosion, e.g. filters over the surface of the foundation.

High groundwater levels, e.g. in alluvial soils, may determine the practical depth to which a cutoff may be taken, since dewatering is usually expensive. The guidelines given below, therefore, are stating the 'desirable' requirements rather than what may be practicable in some cases.

It should also be noted that many of the features described below cannot be readily identified in the base of the cutoff excavation, but are apparent in the sides of the excavation. Hence it is normal to require progressive excavation and cleanup, with a minimum excavation depth of about 0.5 m below general excavation level to confirm that the requirements have been achieved.

11.3.1 *Cutoff in rock*

– Remove rock with open joints, and with other fractures leading to a permeable structure. In many cases this will result in a foundation which has a permeability generally less than say

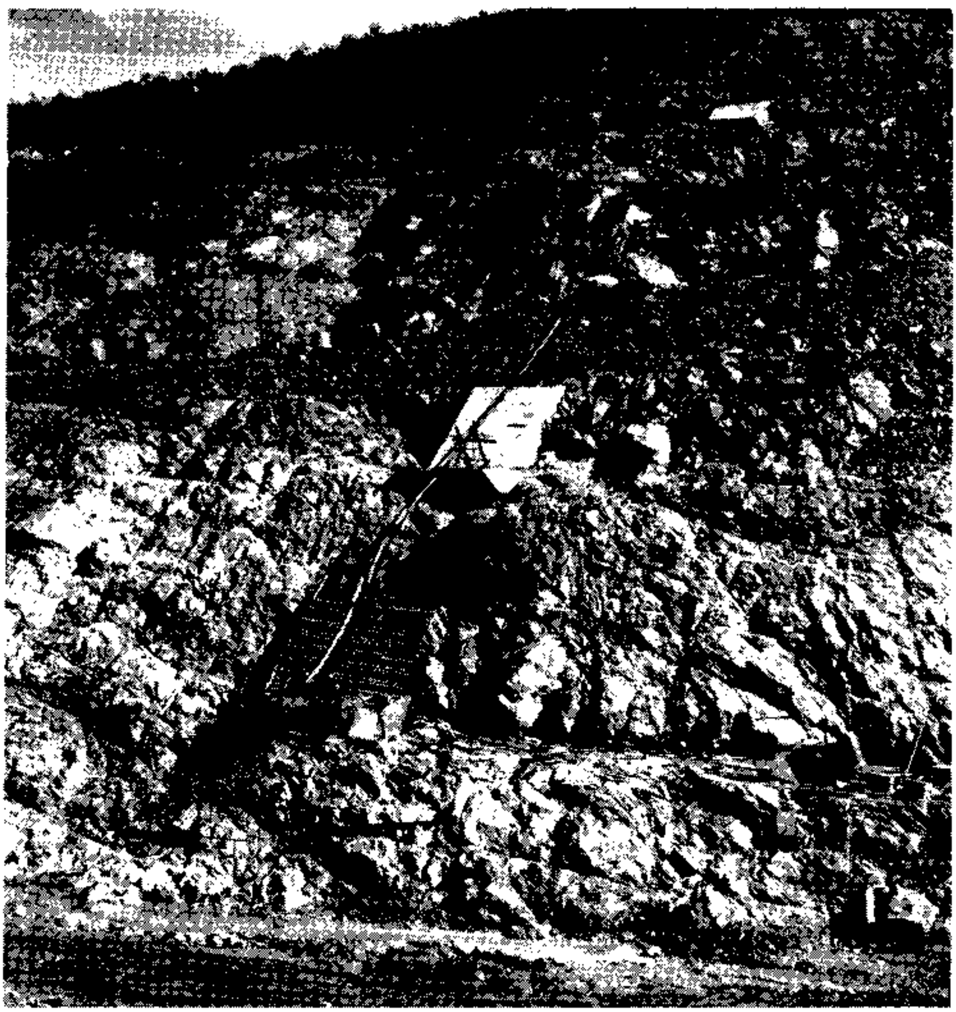


Figure 11.1. Abutment of Dartmouth Dam showing cleanup in cutoff foundation, and construction of the grout cap (courtesy of Rural Water Commission of Victoria).

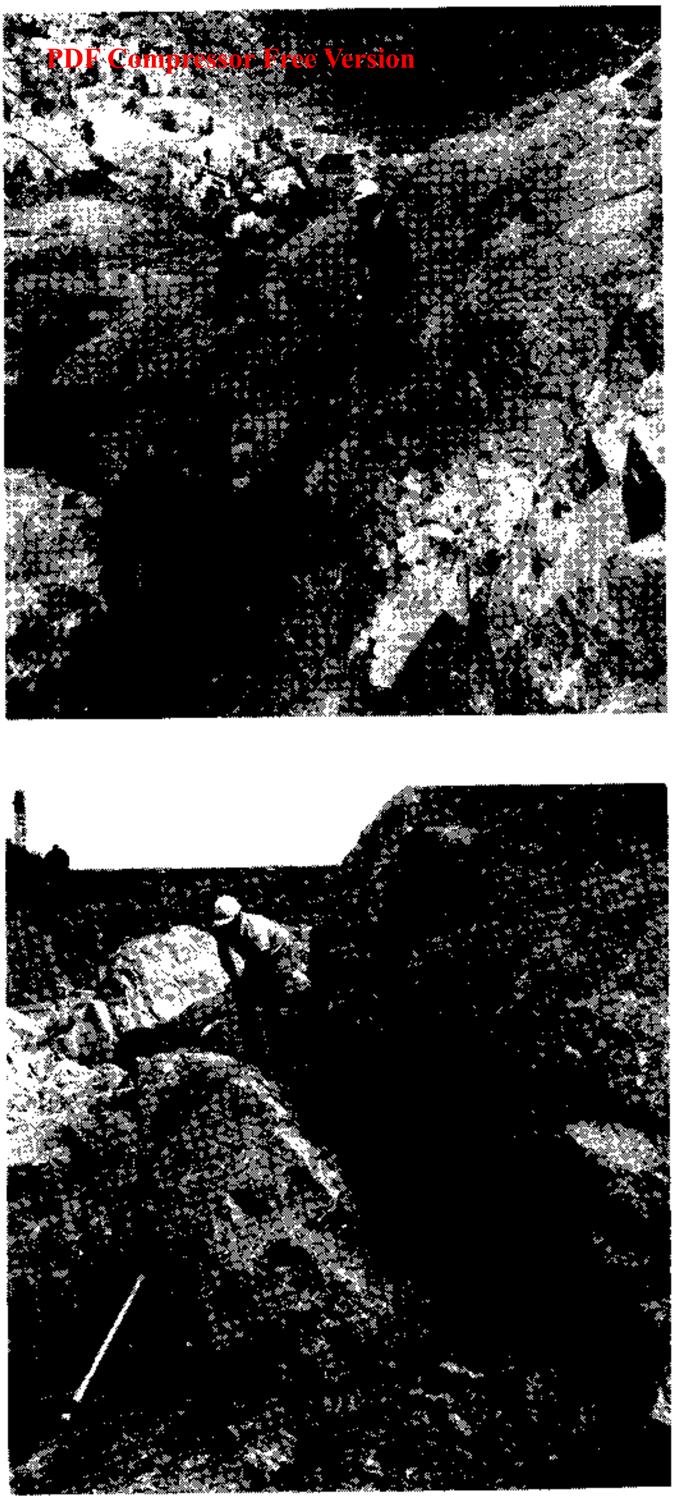


Figure 11.2. Excavation of sheared zones prior to filling with concrete in the cutoff foundation of Dartmouth Dam (courtesy of Rural Water Commission of Victoria).

Figure 11.3. Cutoff foundation of Dartmouth Dam showing excavation and cleanup, pneumatically applied mortar over irregular surfaces and slope modification concrete (courtesy of Rural Water Commission of Victoria).

Lugeons. For large, high hazard dams it would be normal to aim for a foundation with a permeability generally less than 5 Lugeons.

– Remove rock with clay infilled joints, roots etc which may erode under seepage flows to yield a high permeability rock. This is particularly important where the clay has been transported into the joints and/or is dispersive, as these indicate its likely erodibility.

– Carry out slope modification and treatment as described in Section 11.6.

– Where the exposed rock is susceptible to slaking by wetting and drying, eg. many shales, or breakdown under trafficking, it should be covered with a cement-sand grout, pneumatically applied mortar (minimum thickness 50 mm) or concrete. This generally should be done immediately the foundation is exposed, but may be done after a second cleanup immediately prior to placing the earthfill core.

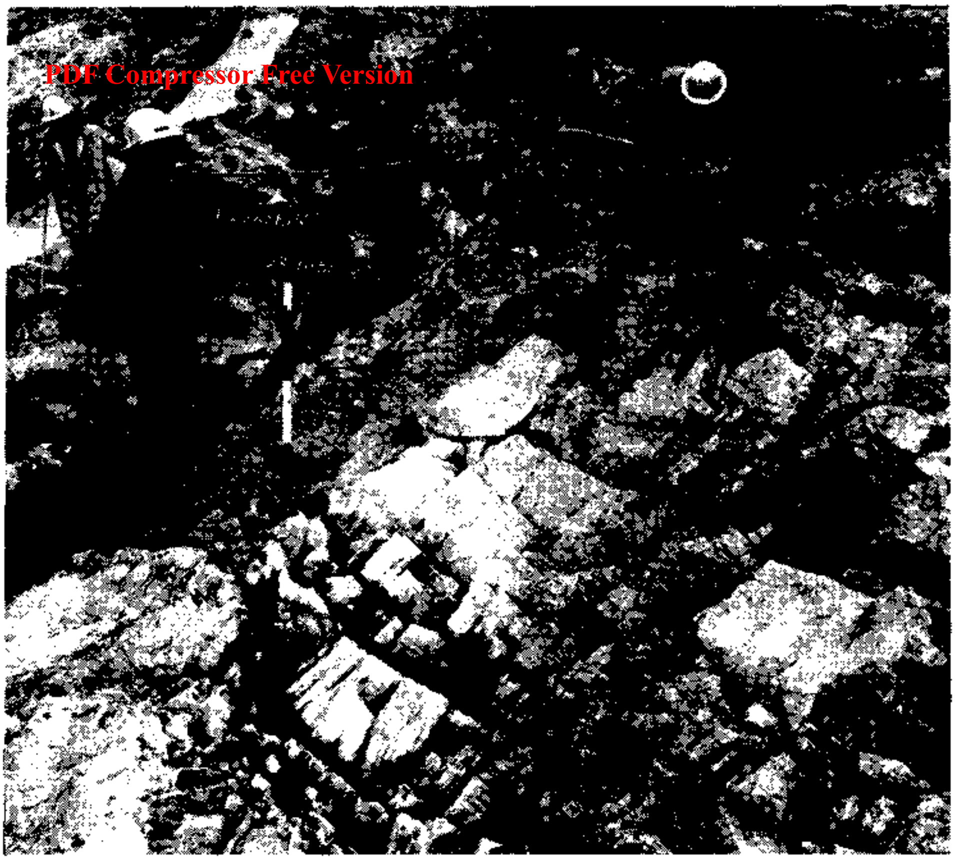


Figure 11.4. Cutoff foundation for Blowering Dam showing open joints being treated with pneumatically applied mortar, but no treatment for other joints (courtesy of Snowy Mountains Hydro-Electric Authority).

– Remove from the surface, using light equipment and with an air or air-water jet, all loose soil and rock, and debris from grouting. Hand cleanup may be necessary. The surface may need to be moistened immediately prior to placing earthfill so as to maintain the earthfill moisture content.

– If the rock in the floor or the sides of the cutoff trench displays open joints or other features which would allow erosion of the earthfill into them, it should be cleaned of loose material and covered by a cement-sand grout, pneumatically applied mortar or concrete. This is particularly critical on the downstream side of the cutoff trench. If there are only a few such features they might be treated dentally.

– Under no circumstances should the surface of the cutoff foundation be rolled, even if it is a weathered rock. Rolling will only disturb the rock leading to a higher permeability material.

Figures 11.1 to 11.6 show cutoff foundation cleanup, treatment of seams and open joints for Dartmouth and Blowering Dams.

Figure 11.5. Application of pneumatically applied mortar on seams in the cutoff foundation of Blowering Dam (courtesy of Snowy Mountains Hydro-Electric Authority).

11.3.2 *Cutoff in soil*

– Remove soil with open fissures, open joints, roots, rootholes, permeable layers (e.g. sand and gravel) and other permeable structure (e.g. leached zones in lateritic soils).

– Remove dispersive soils if possible.

– Carry out slope modification as described in Section 11.6.

– If the soil on the sides of the cutoff trench displays permeable layers or features which would allow erosion of the earthfill into them, it should be trimmed and cleaned, and covered with a filter layer or layers which is designed to control such erosion. In extreme cases it may be necessary to also cover the slope with pneumatically applied mortar. Details of such treatment are shown in Figure 9.7.

– Remove loose and dry soil and other debris, with light equipment, possibly with the aid of an air jet. The base of the cutoff should be watered and rolled before placing the first layer of fill, to compact any soil loosened by the construction work.

Figure 11.6. Left abutment of Blowering Dam after treatment of large fault zone and weathered seam (courtesy of Snowy Mountains Hydro-Electric Authority).

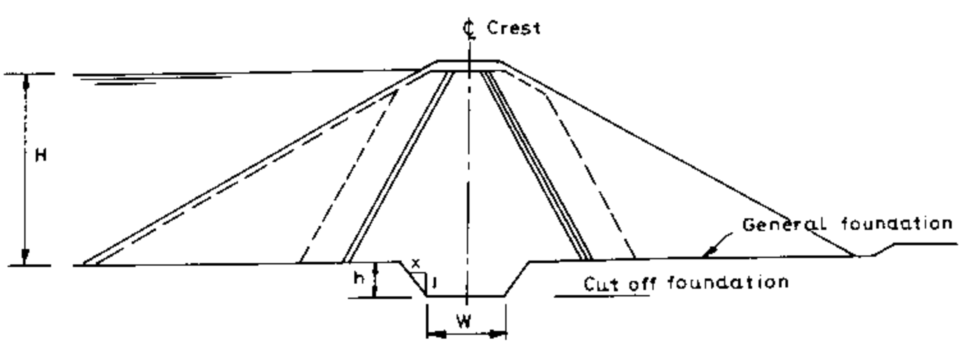


Figure 11.7. Cutoff trench details.

11.4 WIDTH AND BATTER SLOPES FOR CUTOFF

The width of cutoff trench adopted depends on:

– the rock or soil quality at cutoff foundation level,
– the size and hazard rating of the dam,
– the type of embankment.

In general it is not necessary to treat the whole of the contact between the earthfill core and the foundation as 'cutoff,' and only part of that area is excavated to cutoff foundation requirements. Figure 11.7 shows a central core earthfill embankment and for this and other earth, and earth and rockfill embankments the following guidelines are given.

11.4.1 *Cutoff width W*

– For a cutoff in rock, the width depends on the quality of the rock, eg. for low permeability non erodible rock, W/H may be as low as 0.25, and there have been cases where W/H = 0.1 has been used. For concrete face rockfill dams W/H ≈ 0.05 to 0.1 is quite common (Fitzpatrick et al. 1985). For cutoff foundations on more permeable erodible rock W/H may be taken as the full core width, commonly > 0.5H and up to 1H..

– Should be not less than 3 m (the width of excavation and compaction equipment).

– For large high hazard dams it is common to be conservative, and the full core contact is taken to cutoff foundation conditions.

– For a cutoff in soil, it would be common to make W/H ≈ 1. The idea is to give a lower seepage gradient for these more erodible conditions.

It is emphasised that if the depth to an acceptable cutoff is excessive these guidelines may be waived, provided other measures are adopted, e.g. multi-line curtain and consolidation grouting, and erosion protection in the base and sides of the cutoff.

11.4.2 *Batter slope*

The batter slopes for the cutoff trench should be:

– Not steeper than 0.5H:1V, so that the earthfill can be compacted against the sides of the trench.

– Sufficiently flat to avoid arching effects in the cutoff trench. As a guide if h/W > 0.5 say, the batter slopes should be reduced to 1H:1V.

– The batter slopes may be determined by stability during construction, e.g. by joints in rock, or by the strength of the soil. The presence of groundwater will also affect stability. For soil, it would not be uncommon to require batter slopes between 1H:1V and 1.5H:1V and possibly flatter.

11.4.3 *Setting out*

For setting out and construction purposes it is desirable to position the cutoff trench a fixed distance from the dam centreline, or by means of a series of fixed points. Figure 11.8a shows the effect of positioning the trench at the upstream contact of Zone 1 and the foundation. The result is a 'footprint' of the cutoff trench which is curved and more difficult to construct than Figure 11.8b which is positioned relative to the centreline.

The actual width of the base of the trench is best set as a fixed width or widths (narrower up the abutments where the effective height is less).

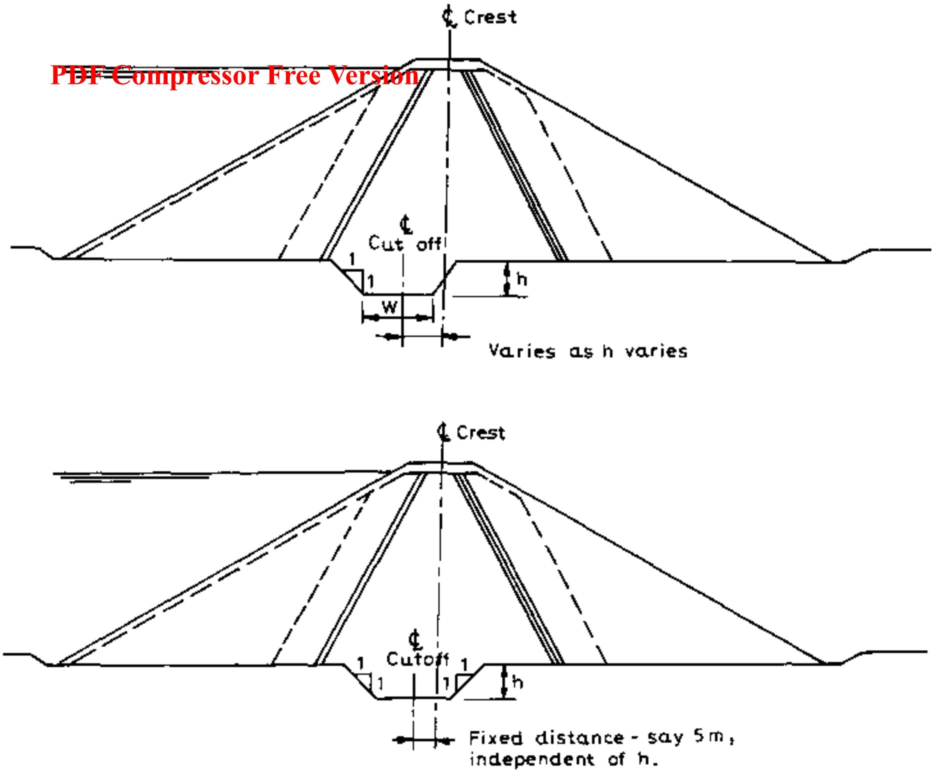


Figure 11.8. Cutoff trench layout.

It should be remembered that the actual depth of the cutoff trench will be determined by geological and geotechnical factors and is not known accurately prior to start of construction. It is necessary to set out the batter pegs for the excavation based on an assumed depth of excavation. It is wise to be somewhat conservative in doing this(particularly if a minimum width (3 m) trench is being used) so that it is not necessary to restart excavation from the surface in the event the actual depth to cutoff foundation is greater than anticipated.

11.5 SELECTION OF CUTOFF FOUNDATION CRITERIA

The selection of cutoff foundation criteria for a project will depend on the particular site and embankment under construction. The following useful hints assist in this process:

– Never specify a depth of cutoff trench i.e. depth below general foundation. The requirements for cutoff are geological and geotechnical, and the depth to achieve this will vary. It is, however, appropriate for the geotechnical engineer/engineering geologist, in consultation with the embankment designer, to estimate the likely depths as a guide for the construction personnel

– Do not specify a weathering grade alone as a requirement to achieve a suitable foundation. The requirements for cutoff foundations are essentially those of permeability and erodibility, and a weathering grade alone (e.g. slightly weathered rock) may not be adequate, or may be too conservative. If weathering grade is used as one of the requirements, it should be clearly defined in the documents what the various weathering grades are

– Planning and selection of cutoff foundation requirements should be done using all available information including, e.g.

borehole log data including rock type, degree of weathering and fracture log,
the Lugeon water pressure tests,
presence of limonite and clay in fill in joints,
seismic refraction data,
bulldozer trench information.

It will often be found that there is some correlation between these data allowing logical decision making, e.g.

– The soil/extremely weathered rock will have low seismic velocity (300-1000 m/sec) and will coincide with blade refusal in the dozer trench. Hence the low seismic layer may form a useful means of extrapolating between 'hard data' in the trenches and boreholes, in developing general excavation requirements.

– The base of highly fractured rock, high-moderate Lugeon value, and medium seismic velocity (1400 to 2000 m/sec, say) will sometimes coincide, and be a reasonable cutoff level. Again the seismic data allows interpolation around the site.

– The base of limonite staining in joints often coincides with the base of high Lugeon rock. If weathering grade is related to this staining there may also be a correlation between weathering grade and Lugeon value.

Selection of cutoff depth is sometimes done by laying out the drill core and the design engineer and engineering geologist observing the core, logs and other data, and selecting the cutoff level or levels there and then. Often there will be more than one choice, with a 'better' cutoff being available at depth, and an 'adequate' cutoff closer to the surface. The 'adequate' cutoff may require a wider cutoff trench, more grouting, a grout cap, downstream filter protection etc, the need for which has to be offset against the shallower depth.

If this approach is used, the geological features which were used to select the cutoff can be described, and readily identified in the construction process. Ideally, the engineering geologist and design engineer who make the design decision, should be present to define and confirm those conditions in the cutoff trench during construction.

11.6 SLOPE MODIFICATION AND SEAM TREATMENT

11.6.1 *Slope modification*

Excavation of near vertical or overhanging surfaces, and/or backfilling with concrete is required.

For general foundation:

– to allow compaction of earthfill and avoid cavities under rockfill due to overhangs in rock in the abutment.

For cutoff foundation:

– to allow compaction of earthfill;

– to allow maintenance of positive pressure of the earthfill on the abutment;

– to limit cracking of the earth core due to differential settlement over large discontinuities in the abutments, e.g. as shown in Figure 11.9.

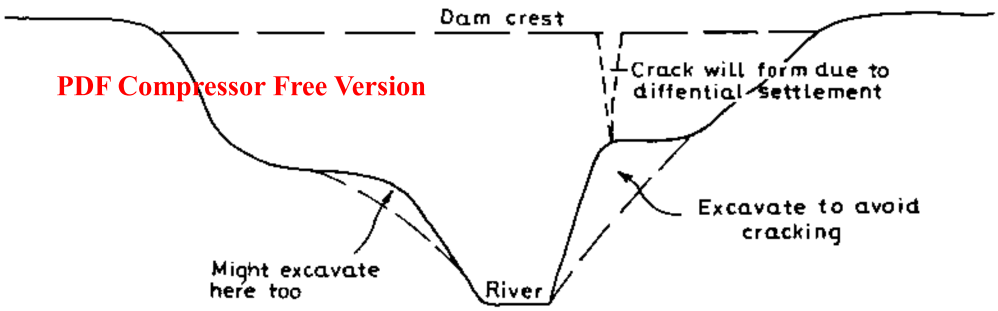


Figure 11.9. Slope modification to reduce differential settlement and cracking of the earthfill core.

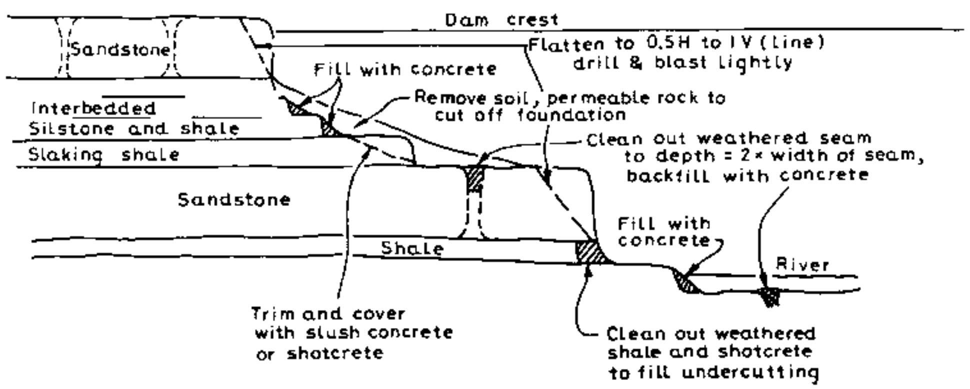


Figure 11.10. Slope modification in the cutoff foundation for interbedded sandstone and siltstone, near horizontal bedding.

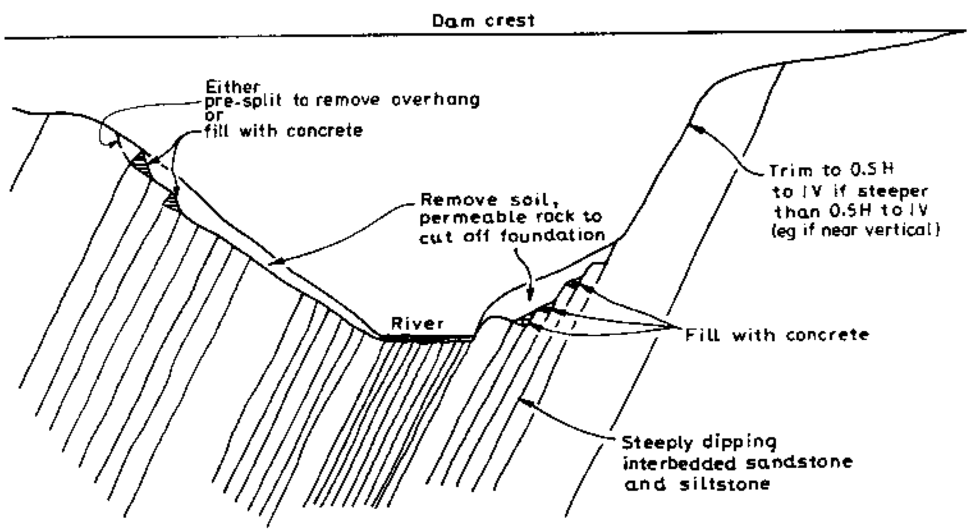


Figure 11.11. Slope modification in the cutoff foundation for interbedded sandstone and siltstone, steeply dipping bedding.

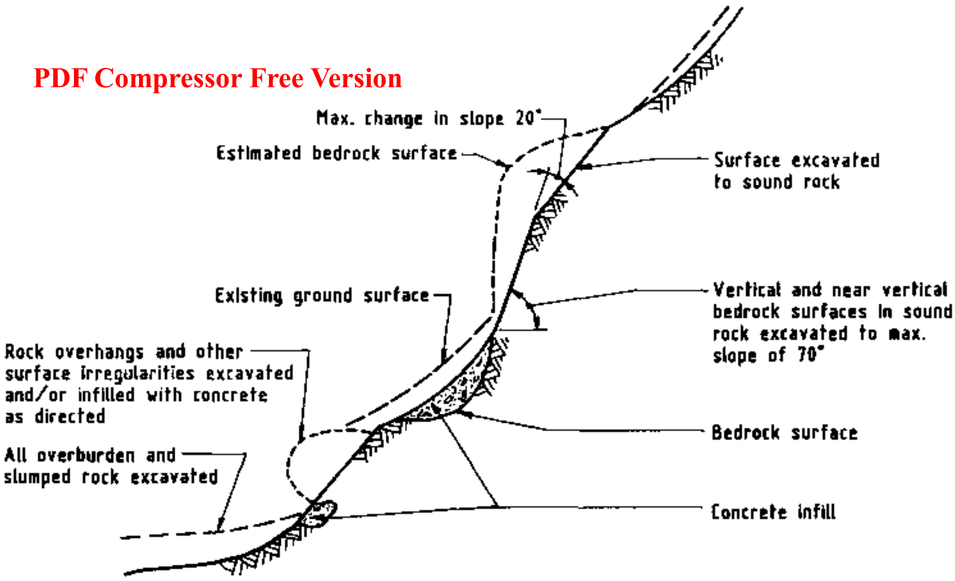


Figure 11.12. Mica Dam foundation excavation, typical excavation detail (Pratt et al. 1972).

To allow earthfill to be compacted and maintain positive pressure on the abutments, steep foundation surfaces should be flattened to about 0.5H to 1V either by excavation, or by backfilling with concrete. Figures 11.10 and 11.11 give examples of this type of treatment. Some authorities require a flatter slope, e.g. Thomas (1976) and Wallace & Hilton (1972) suggest the use of 0.75H:1V. Others, e.g. Acker & Jones (1972), accept steeper slopes (0.25H:1V). Thomas and Wallace and Hilton are considering very large dams and it might be argued greater conservatism is warranted in these cases. The authors are of the opinion that 0.5H:1V is satisfactory provided the earthfill is compacted wet of optimum with rubber tyred equipment to 'squeeze' it into position.

In some geological environments these requirements for slope modification may necessitate virtually the whole of the cutoff area being excavated or covered with concrete. It should be noted that in some cases there will be tendency for the rock to pluck out on joints, repeating the oversteepening problem. In these cases backfill concrete, or possibly presplitting with light blasting may be necessary.

Large scale slope modification to avoid differential settlement and resultant cracking of the earthfill core has been used on many dams, e.g. Pratt et al. (1972) describe slope modification for Mica and Bennett Dams in Canada. The work carried out is shown in Figures 11.12 and 11.13.

Walker & Bock (1972) give details of slope correction work at Blue Mesa Dam. Details are shown in Figure 11.14.

In this case some of the slope correction work was required to remove loose unstable rock from the abutments.

Since these projects were constructed it has been recognized that earthfill core for dams may crack even in ideal conditions, and the best line of defence is to provide good filters (see Chapter 7). Hence the desirability of such major slope correction works is less clear. The authors'

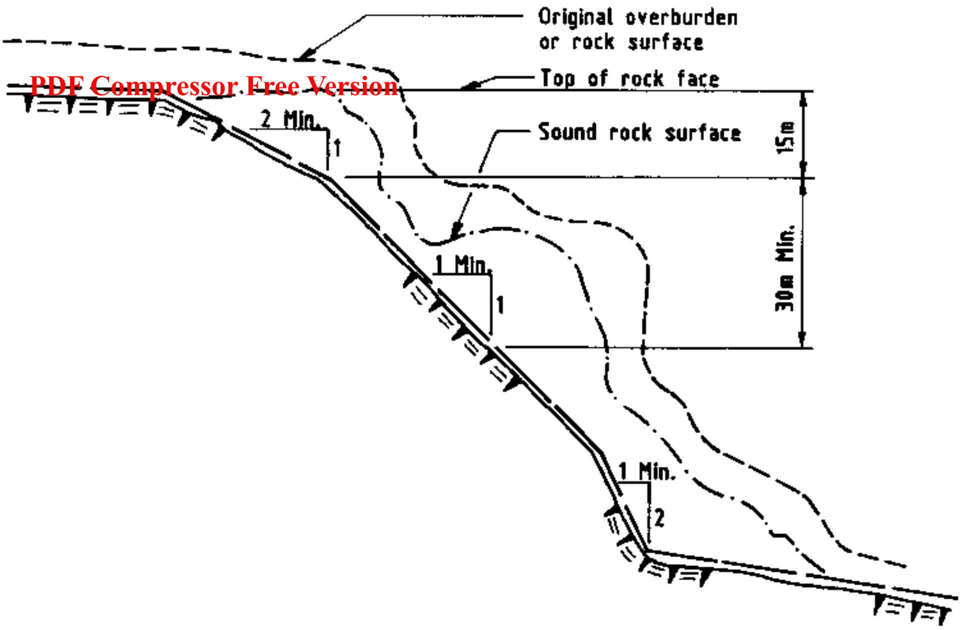


Figure 11.13. Bennet Dam, typical core abutment excavation requirements (Pratt et al. 1972).

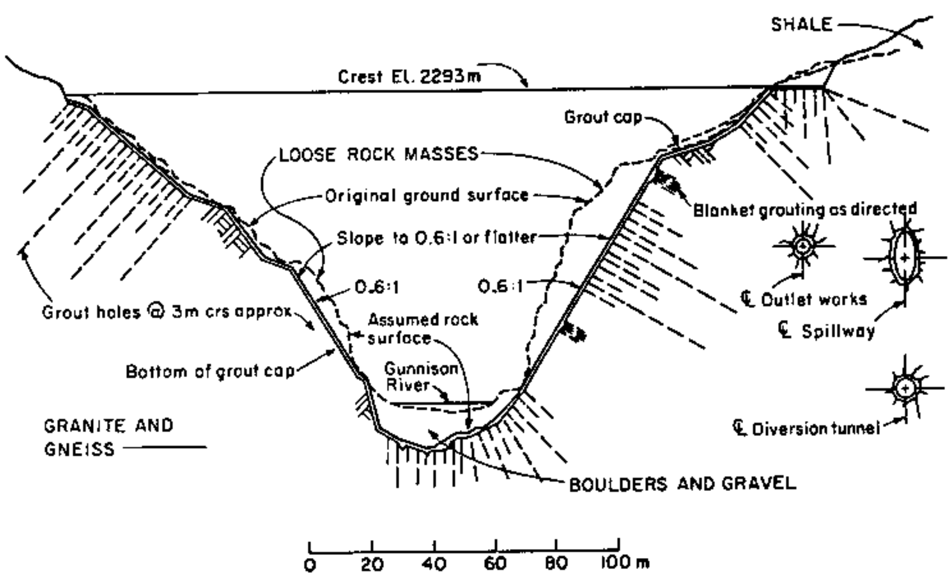


Figure 11.14. Foundation slope correction, Blue Mesa Dam (reproduced from Thomas 1976). Reprinted by permission of John Wiley & Sons Inc.

Figure 11.15. Slope modification in the cutoff foundation of Dartmouth Dam (courtesy of Rural Water Commission of Victoria).

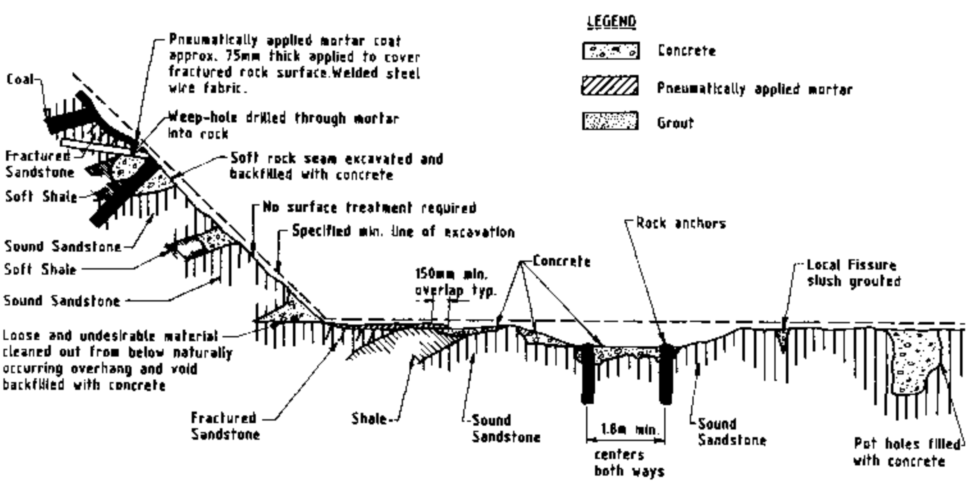


Figure 11.16. Typical core contact surface treatment details, Mica Dam (Pratt et al. 1972).

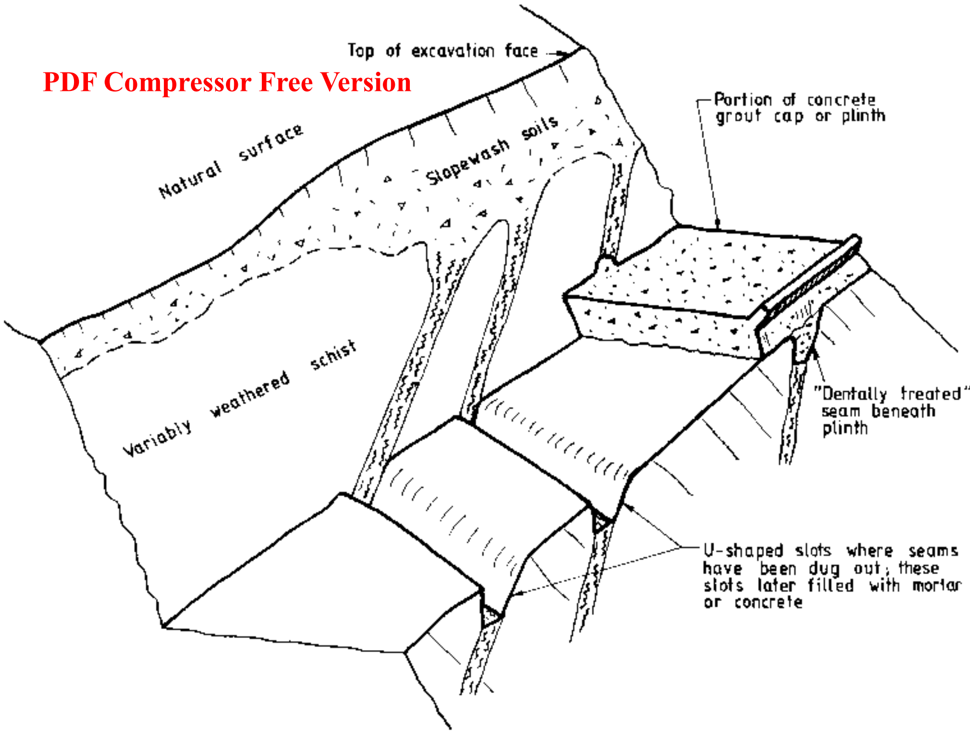


Figure 11.17. Dental treatment of weak seams in the plinth foundation of Kangaroo Creek Dam.

assessment is that provided the overall slope is less than 0.25H:1V (preferably 0.5H:1V) and good filters are provided, one should not be too concerned about the large scale slope modification carried out in the examples above. However, it is necessary to avoid local steep slopes and overhangs against which earthfill is to be compacted.

Figure 11.15 shows such treatment on Dartmouth Dam.

A more conservative approach to large scale modification would apply for dams in severe earthquake areas and for very large dams.

11.6.2 *Seam treatment*

It is common practice to require that seams of clay or extremely weathered rock which occur in the cutoff foundation should be excavated and filled with concrete. This is done to avoid erosion of the seams allowing seepage to bypass the earth core and filters.

Thomas (1974) suggests that the depth of excavation and backfill should be 2 to 3 times the width of the seam; Wallace & Hilton (1972) indicate that for talbingo dams all seams wider than than 12 mm were excavated to a depth equal to the width and filled with concrete, or the area covered with thick grout, pneumatically applied mortar or concrete. This latter approach seems more reasonable for narrow seams. Pratt et al. (1972) give the detail shown in Figure 11.16 for Mica Dam.

Figure 11.17 shows seam treatment adopted for Kangaroo Creek Dam, and Figure 11.2 that adopted for Dartmouth Dam.

These examples can be used as a guide to reasonable practice. The authors would not be so keen to dig out the seams, as to ensure that they are adequately covered with concrete or grout, since their nature is not likely to change with one or two widths, and the concrete will be held in place by the earthfill.

For foundations with continuous seams filled with dispersive or erodible soil and rock, a horizontal filter drain may be needed downstream of the earthfill core to allow foundation seepage to emerge in a controlled manner.

CHAPTER 12

Foundation grouting

12.1 GENERAL CONCEPTS OF GROUTING DAM FOUNDATIONS

The foundations for most dams more than 15 m high, and for some which are smaller, are treated by grouting. Grouting consists of drilling a line or lines of drill holes from the cutoff level of the dam into the dam foundation, and forcing cement slurry, or chemicals into the foundation under pressure. Figure 12.1 shows an example:

The grouting is carried out to:

– reduce leakage through the dam foundation,

– reduce seepage erosion potential,

– reduce uplift pressures (under concrete gravity dams when used in conjunction with drain holes,

– strengthen the dam foundation and reduce settlements in the foundation (for concrete gravity, buttress and arch dams).

Most foundation grouting is done using cement grout: Portland cement mixed with water in a high speed mixer to a water-cement ratio (mass water/mass cement) of between 0.5 and 5 at which condition it is a slurry, readily pumpable and able to penetrate fractures in the rock in the dam foundation.

If the dam is on a soil foundation (e.g. sand), or if the fractures in the rock are very narrow, chemicals can be used instead of cement. These tend to be more expensive than cement grout so they are only used where cement grout would not be successful.

Foundation grouting takes two forms:

– curtain grouting,

– consolidation grouting.

Curtain grouting is designed to create a thin barrier (or curtain) through an area of high permeability. It usually consists of a single row of grout holes which are drilled and grouted to the base of the permeable rock, or to such depths that acceptable hydraulic gradients are achieved. For large dams on rock foundations, or dams on very permeable rock or soil foundations, 3 or 5 or even more lines of grout holes may be adopted.

The holes are drilled and grouted in sequence to allow testing of the permeability of the foundation (by packer testing) before grouting and to allow a later check on the effectiveness of grouting from the amount of grout accepted by the foundation ('grout take'). Thus in Figure 12.1 primary holes are drilled first, followed by secondary, and then tertiary. The final hole spacing will commonly be 1.5 or 3 m, but may be as close as to 0.5 m. This staged approach allows control over the amount and effectiveness of the grouting.

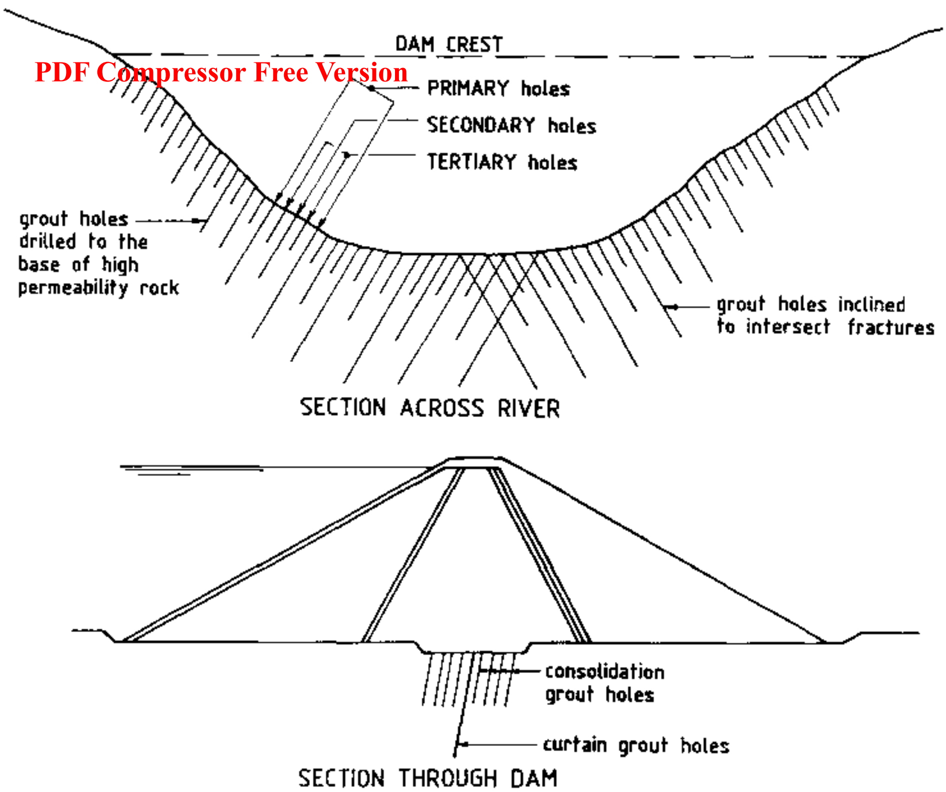


Figure 12.1. An example of dam foundation grouting.

Consolidation grouting is designed to give intensive grouting of the upper layer of more fractured rock, in the vicinity of the dam core, or in regions of 'high' hydraulic seepage gradient, eg. under the plinth for a concrete face rockfill dam. It is usually restricted to the upper 5 to 15 m. While carried out in sequence, consolidation grouting is commonly to a predetermined hole spacing.

12.2 GROUTING DESIGN – CEMENT GROUT

12.2.1 *Staging of grouting*

Grouting of holes is normally carried out in stages, the method depending on the permeability and quality of the rock being grouted, and the degree to which control of the grouting operation is desired. Figures 12.2, 12.3, 12.4, 12.5 and 12.6 show the various methods available.

Downstage without Packer (Fig. 12.2). This is one of the preferred methods for high standard grouting, since each stage is drilled and grouted before the next, lower stage, allowing progressive assessment as to whether the hole has reached the desired closure requirement. This method reduces the risks of leakage of grout to the top stage and also allows higher pressures to

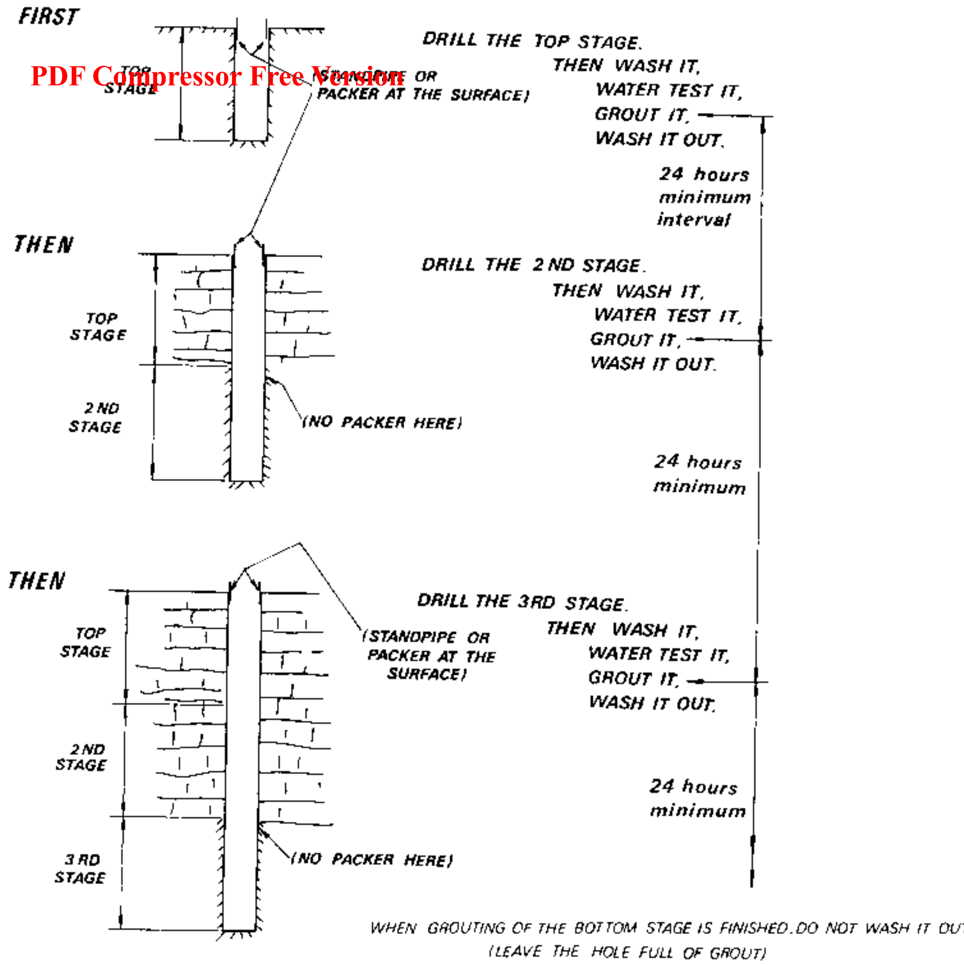


Figure 12.2. Grouting downstage without packer (Water Resources Commission 1981).

be used for lower stages, giving better penetration between holes. The grout pressures are limited by the effectiveness of the top stage grouting. It does necessitate a separate set up of the drill for each stage and separate 'hookups' of the grout lines. It is, therefore, relatively expensive. This method is the one preferred by Houlsby ((1977, 1978, 1982).

Downstage with Packer (Fig. 12.3). This has the attributes of the above method, and in addition allows use of increased grout pressures for lower stages, since these pressures are not applied from the surface. However, there may be problems with seating and leakage past the packer. Bleeding of the grout hole (i.e. removing the 'clear' water which accumulates at the top of the grout hole as the cement settles) cannot be achieved except at the ground surface (i.e. not immediately above the grout stage). Ewart (1985) indicates a preference for this method, because of the potential to fracture the rock in the upper levels if downstage without packer methods are used.

Upstage (Fig. 12.4). Does not allow progressive assessment of the depth of grout hole

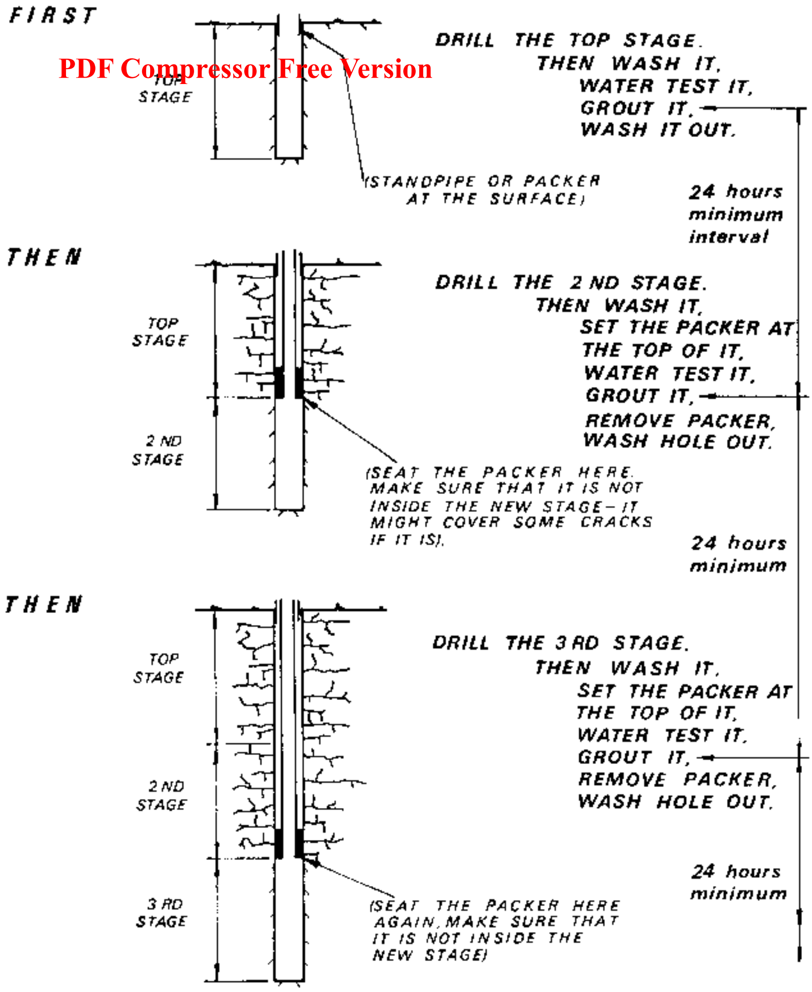


Figure 12.3. Grouting downstage with packer (Water Resources Commission 1981).

needed to reach a desired closure requirement as the holes are drilled to their full depth in one stage. The method is cheaper in principle than downstage methods since the drill rig is only set up once, but these savings may be offset by the need for more conservative total depths. The method is susceptible to problems with holes collapsing, or over enlarging during drilling and grouting in poor rock conditions, making seating of packer difficult or impossible. It is also subject to the same problems as the downstage with packer method regarding bleeding. The method is appropriate for secondary or tertiary holes when depths are reasonably well known, and in strong rock with holes not likely to collapse or erode.

Full Depth (Fig. 12.5). Does not allow proper assessment of where grout take is occurring, or proper monitoring of reduction in Lugeon values with grouting. Depths are predetermined so the method does not allow logical assessment of grout depth based on closure requirements. Grouting pressures are limited. It is not an acceptable method except for consolidation grout holes.

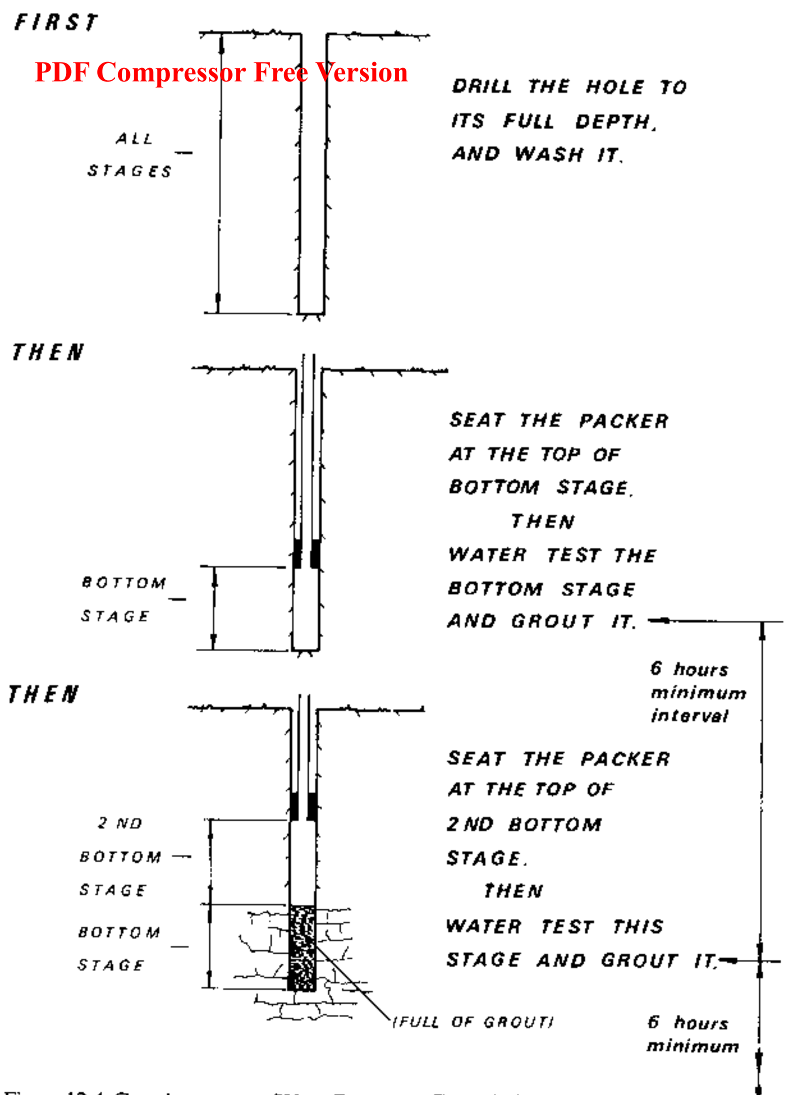


Figure 12.4. Grouting upstage (Water Resources Commission 1981).

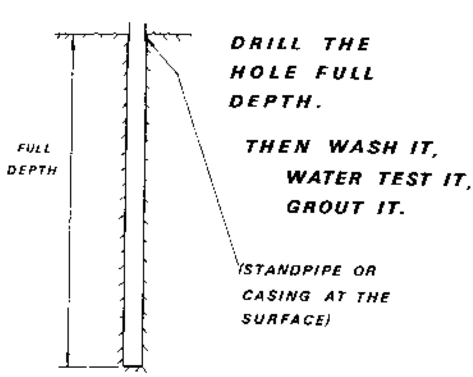


Figure 12.5. Grouting full depth (Water Resources Commission 1981).

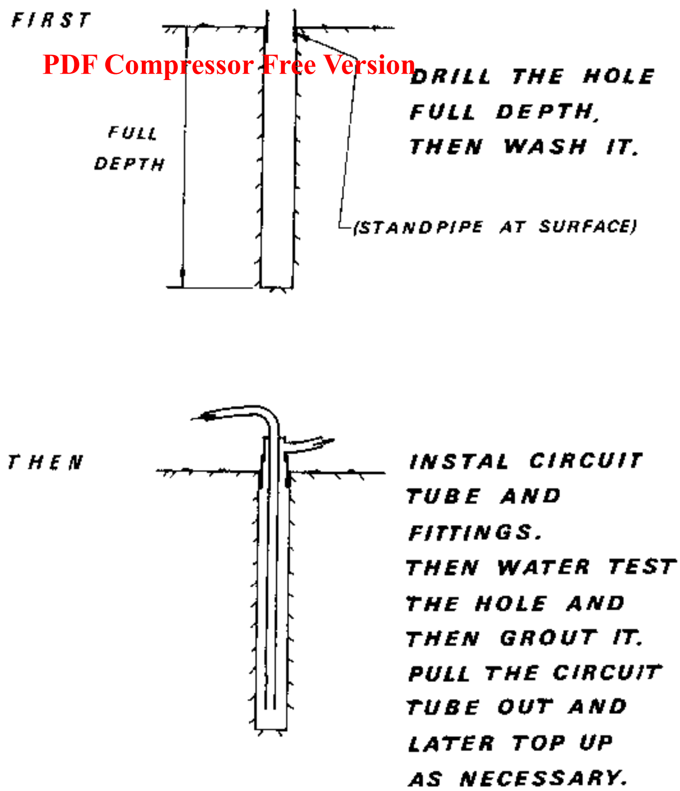


Figure 12.6. Grouting full depth, circuit (Water Resources Commission 1981).

Full Depth Circuit (Fig. 12.6). Has the same limitations as full depth grouting, but by injecting the grout into the base of the hole, the possibility of settled grout is lowered.

For stage grouting, the stage lengths are commonly predetermined depending on:

– the geological conditions, and depths at which changes in degree of permeability are likely to occur,

– the minimum length worth drilling, since short stages are more costly to drill because of set up costs,

– allowable pressures in the upper part of the hole (dependent on geological conditions).

Commonly grout stages will be 5 to 8 m but may be increased in length lower in the foundation, e.g. Houlsby (1977) suggests:

Stage	Depth Range (m)
1	0 to 8
2	8 to 16
3	16 to 30
4	30 to 50

While this reduces the number of drill setups and grout hookups, it may result in unnecessarily deep holes, particularly for smaller dams.

Smaller than the predetermined stage lengths should be used when

– drilling water is lost, indicating a relatively large fracture or opening has been encountered;

– the grout hole is caving, due to, for example, closely fractured rock;

– water flows into the hole under pressure;

– very large water pressure test or grout takes are encountered (often it is possible to relate these to a specific geological feature).

12.2.2 *The principles of 'closure'*

Apart from consolidation grouting, which may be carried out to a predetermined depth and hole spacing, grouting should be carried out sequentially to achieve a predetermined standard of water tightness. This will usually require the successive halving of hole spacing from primary to secondary to tertiary holes etc as shown in Figure 12.1. Whether the required standard has been achieved will normally be determined on the basis of water pressure test Lugeon value on the grout stage prior to grouting and/or on grout take – the volume (or weight of cement) of grout per metre of grout hole. Closure criteria will be discussed in more detail below. Figure 12.7 shows the basic principles of hole closure i.e. that secondary holes are drilled halfway between primary if water pressure tests (and/or grout take) or the primary holes fail to meet the closure criteria.

Figure 12.8, taken from Houlsby (1977) and Water Resources Commission (1981), gives examples of the closure method when Lugeon water pressure test values are the required standard.

In these examples:

Case (a). Primary grouting has resulted in a reduction in Lugeon value (and grout take) in the secondary hole, and tertiary grouting has resulted in further reductions close enough to the closure requirement of 7 Lugeons. Hence no further grouting is required. If the closure requirement was say 5 Lugeons, quaternary holes would have been required, and would seem worthwhile as there is a progressive reduction in Lugeon value and in grout take.

Case (b). Primary and secondary grouting has resulted in no significant reduction in Lugeon value in the tertiary holes prior to grouting them. Houlsby (1977) concludes that quaternary holes are needed and may be quinary later. If holes are already at say 1.5 m spacing, considera-

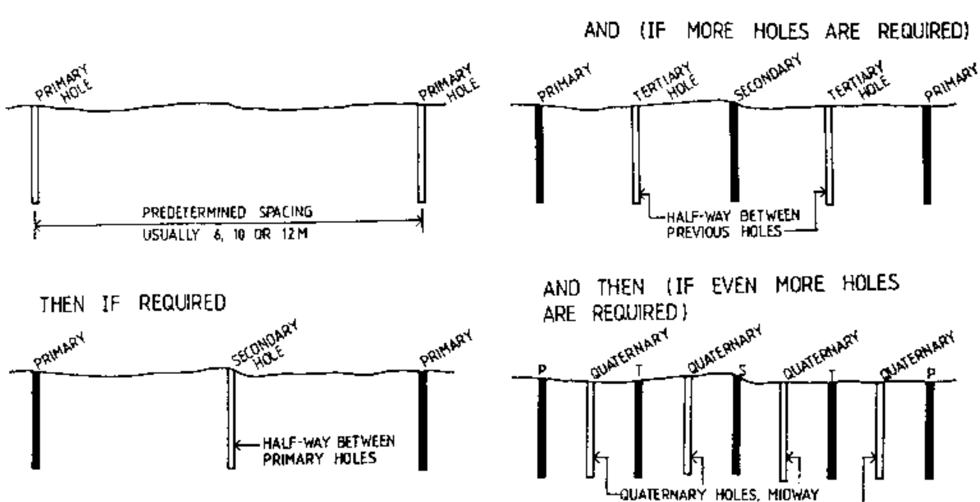


Figure 12.7. Basic concept of halving hole spacing to achieve closure.

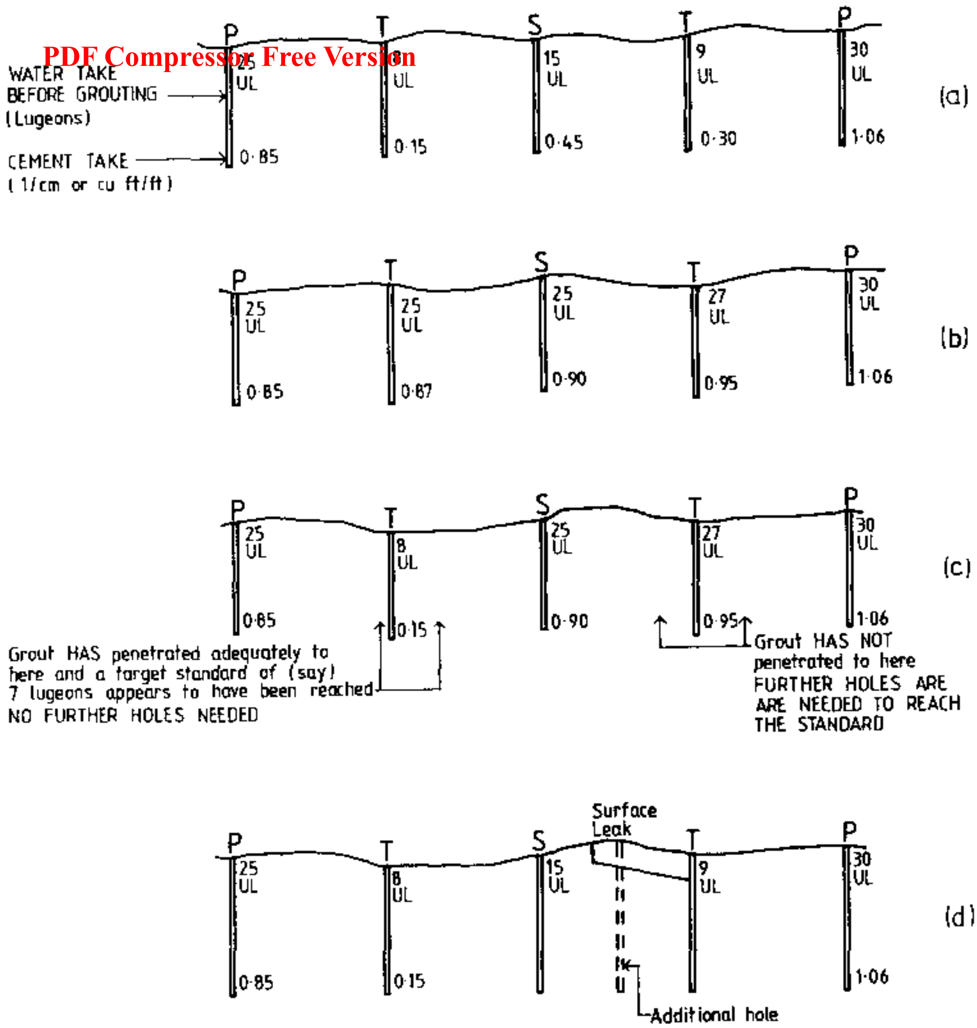


Figure 12.8. Examples of grout closure based on Lugeon water pressure test (and grout take as secondary information) – adapted from Houlsby (1977) and Water Resources Commission (1981).

tion should also be given to whether grouting with cement is having any significant effect. If it is concluded that cement grouting is ineffective then grouting might be discontinued on the basis that it is not necessary, or else it might be continued using chemical grout.

Case (c). A closure criterion of 7 Lugeons has been achieved in part, but quaternary holes are required elsewhere. Since closure is being achieved in part it would seem reasonable to proceed to this quaternary stage of grouting where needed.

Case (d). As for Case (a) except that leakage to the surface occurred in grouting the tertiary hole, necessitating further grouting to seal the leak.

Closure requirements may vary with depth, and the hole spacing and depth required to achieve closure may also vary considerably. Figure 12.9 shows an example taken from Water

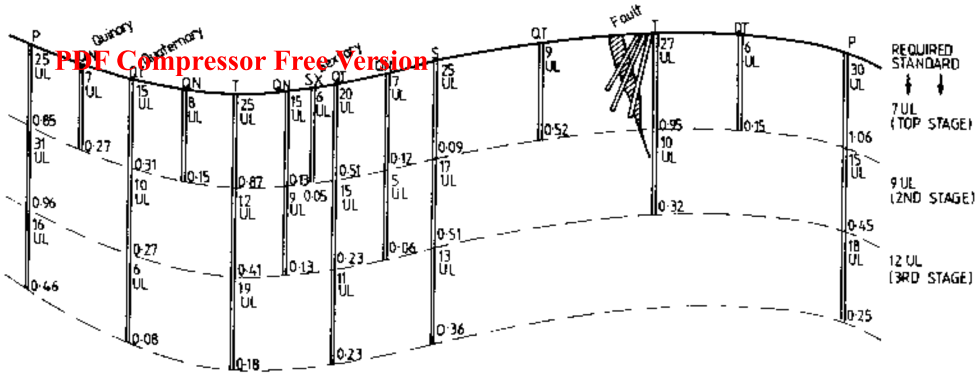


Figure 12.9. Example of closure based on water pressure test Lugeon criteria (adapted from Houlsby 1977 and Water Resources Commission 1981).

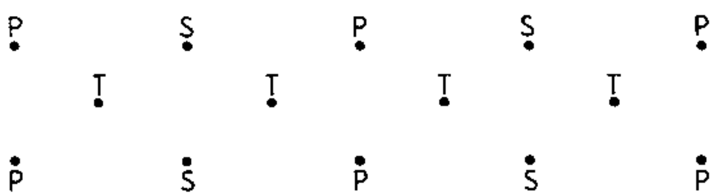


Figure 12.10. Grout closure with 3 lines of grout holes.

Resources Commission (1981). Note that this is a particularly thorough example of grouting, where if the primary spacing was 12 m, the final spacing is 0.4 m in some areas. As discussed below, it is unlikely that in most cases this method is warranted.

When more than one line of grout holes is planned, the holes should be drilled and grouted in sequence to form the outer lines ahead of the central line, so that the progressive development of closure can be observed. Figure 12.10 shows a possible closure sequence.

12.2.3 *Closure criteria*

There are several different opinions on how to define acceptable closure criteria for grout curtains, and on how to select the limits. This is critical to the question as to whether sufficient grouting has been carried out. Houlsby (1977, 1978, 1982) and Water Resources Commission (1981) propose the use of the Lugeon water pressure test values in the grout holes prior to grouting as a basis for decision making. Figure 12.11 shows the proposed criteria. Houlsby qualifies these criteria as being a guide only, and points out that cracking frequency and size must be considered. Houlsby points out that such permeability based criteria originated with Lugeon (1933) who suggested for dams over 30 m high, grouting should continue until the pre-grouting Lugeon value was less than 1 (less than 3 for dams less than 30 m high).

In a later paper, Houlsby (1985) gives revised criteria which are marginally less conservative (see Table 12.1).

Deere (1982) suggests a closure method based on grout 'takes' i.e. the amount of grout absorbed into the rock per metre of grout holes. These are given in Table 12.2.

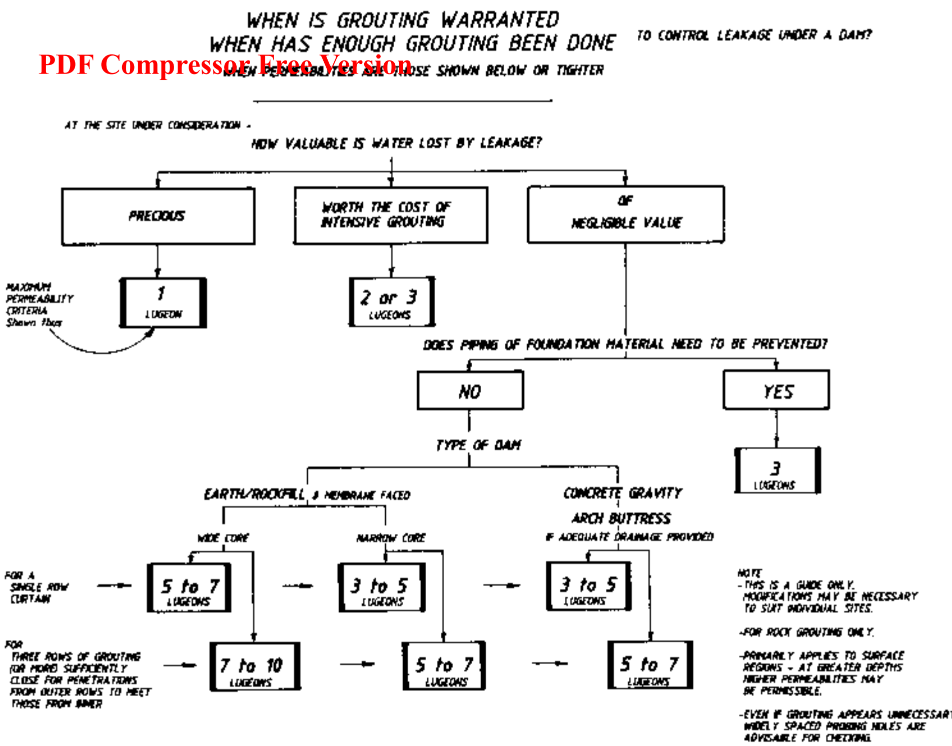


Figure 12.11. Closure criteria proposed by Houlsby (1977, 1978, 1982) and Water Resources Commission (1981).

Where grout absorptions in the secondary holes are greater than the limits quoted, tertiary holes are required, and if grout takes in the tertiary holes exceed the limits, quaternary holes would normally be required.

Bruce (1982) in summarizing British practice indicates that the following criteria were used for three dams.

Wimbleball. Take ≯ 25 kg/m average in last stage, and > 3 Lugeons (apparently before last stage grouting).

Kielder. Take ≯ 25 kg/m average for the last stage, and ≯ 10 Lugeons measured in separate holes after grouting.

Grunworth. Not more than 10 Lugeons after grouting.

The Swiss National Committee on Large Dams (1985) indicated that the earlier practice of requiring closure to 1 Lugeon has been abolished, and grouting is now generally ceased when grout take is less than 50 kg/m.

Bozovic (1985) in the general report on foundation treatment for control of seepage at the XVth Congress on Large Dams concluded that there was general concensus that the correlation between Lugeon value and grout take is very weak. This is discussed further in Section 12.2.4.

Such poor correlation has been observed by many authors including Houlsby (1982).

Ewart (1985) discusses at some length whether water pressure test Lugeon values are a valid criterion for closure and concludes that alone they are not, largely because of the poor correlation between grout take and Lugeon value.

Table 12.1. Revised closure criteria proposed by Houlsby (1985).

General case	Suggested curtain standard (Lugeons)
Concrete dams (gravity, arch, buttress)	
Single row curtain	3 to 5
Multiple row curtain	5 to 7
Embankment dams	
Narrow core earth/rockfill	3 to 5
Wide core earth/rockfill, and membrane faced	
Single row curtain	5 to 10
Multiple row curtain	7 to 15
Exceptions	
All types of dam if foundation contains material able to be removed by seepage:	
Single row curtain	3
Multiple	4
All types of dam if water lost by seepage is sufficiently valuable to warrant considerable expenditure to stop it, or is environmentally hazardous:	
Single and multiple row curtains	1 to 3

The standards suggested above are for guidance by departures from them in appropriate circumstances are made. The standards mainly refer to the top stage of holes; lower stages may be more permeable taking cognizance of the longer seepage paths through them. Where ranges are given, the lower figure refers to the deeper part of the valley and the lesser to higher parts or to relatively low dams. Difficult geology may warrant more widespread use of the lower end of the range than favourable geology does. Specific foundation weaknesses, such as faults, shears, etc. require specific concentrated grouting, usually to a locally better standard than the majority of the curtain.

Table 12.2. Upper limits of grout absorptions for secondary holes as a function of depth suggested by Deere (1982).

Depth interval (m)	Grout absorption (kg/m of hole)
0-10	25
10-20	35
20-30	50
> 30	100

He suggests that:

– Rock with a Lugeon value < 5 Lugeon is virtually ungroutable with cement grout because the fractures are too fine.

– Rock with a Lugeon value of 10 Lugeon may be ungroutable if this water take is due to many fractures, e.g. 6 fractures per metre.

– As far as practicable, the Lugeon value should be considered in association with the number of fractures, and the limiting value adjusted accordingly.

– The allowable grout pressure (i.e. not leading to fracture) also has an influence on whether existing fractures can be grouted.

Details can be found in Section 8.3.5 of Ewart (1985).

12.2.4 *Effect of cement particle size, viscosity, fracture spacing and Lugeon value on the effectiveness of grouting*

In an attempt to rationalise the logic in selecting closure criteria, one of the authors has investigated the effect of cement particle size and viscosity, and fracture spacing and opening on:

a) whether a cement grout will penetrate into fractured rock;

b) if so, how far away from the grout hole;

c) what effect this will have in reducing seepage under a dam.

This work is described in Fell & MacGregor (1986), Fell et al. (1989), Hawkins (1988) and Tjandrajana (1989). The following sections summarize the results of these investigations.

12.2.4.1 *Particle size*

Figure 12.12 shows the particle size of the dry powder form of Type A and Type C (sulphate resistant) Portland cements manufactured according to AS1315-1982 and microfine cement-ONODA Alofix MC-500 (a portland/slag cement manufactured in Japan). Type A cement is roughly equivalent to ASTM Type I, Type C to ASTM Type IV. These particle size distributions are determined for the 'dry powder' using a CILAS granulometer, a technique in which the cement powder is suspended in alcohol and the sizing determined by detecting diffraction of a laser beam. Also shown for comparative purposes is the particle size analysis for microfine cement shown in Clarke (1984).

The chemical properties of the cements are given in Table 12.3

To assess the actual particle size distribution of the cement grout in suspension a series of hydrometer and microvideomat particle size analysis tests were carried out.

The results shown in Figure 12.12 are for 2:1 water cement ratio (by weight). Tests with water cement ratio ranging from 5 to 1 to 0.5 to 1 showed that the different proportions did not

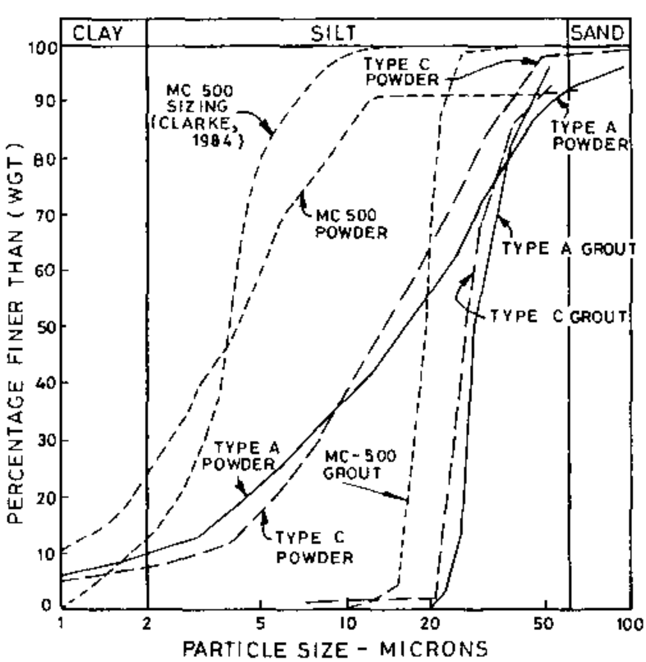


Figure 12.12. Cement powder and 2:1 water cement grout particle size.

Table 12.3. Chemical analysis of cements used in experiments.

Chemical properties	Type 'A' %	Type 'C' %	MC500 %
SiO_2	19.4	22.5	27.2
Al_2O_3	5.4	4.4	9.7
TiO_2	0.3	0.2	0.9
Fe_2O_3	3.7	4.9	1.2
Cao	63.8	62.6	49.0
MgO	1.2	0.86	5.6
SO_3	2.4	2.3	2.7
K_2O	0.49	0.36	0.35
Loss on ignition	3.6	1.8	1.8
Total	100.29	99.92	98.45
Hypothetical compound composition %			
C_3S	64	41	N/A
C_2S	8	34	N/A
C_3A	8.1	3.4	N/A
C_1AF	11	15	N/A
Free Ca (%)	0.1	0.1	3.4
Fineness index (M^2/kg)	335	360	710

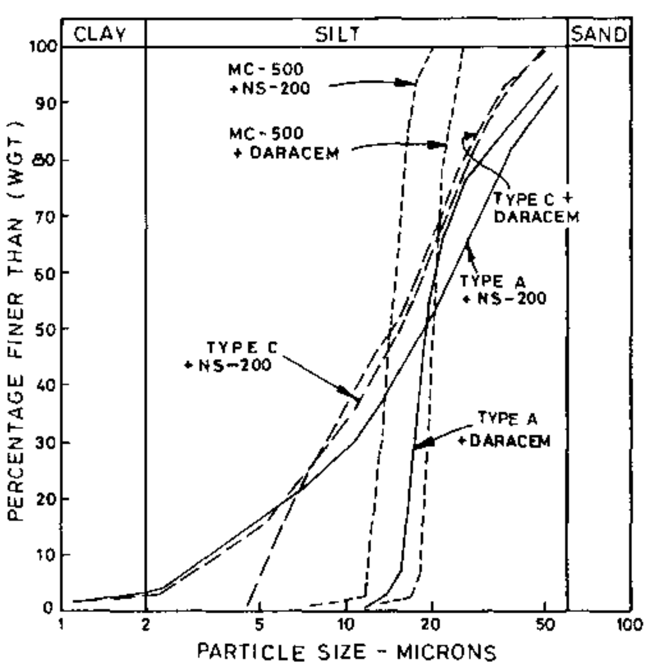


Figure 12.13. 2:1 Water cement ratio grouts with dispersants.

have a significant effect. Mixing in a slower speed mixer for up to 30 minutes (to simulate mixing in a holding tank) showed no significant difference in particle size except that there was some breakup of coarser particles.

It can be seen that in the grout mix, the finer cement particles aggregate together in water giving a coarser effective size than the dry cement powder. Also Type C (sulphate resistant) cement grout is finer than Type A, and the MC-500 microfine cement grout is significantly finer than the conventional cement grouts.

To gauge the effect of adding dispersants to the grout, DARACEM superplasticizer, and ONODA NS-200 dispersant (supplied by the manufacturer for use with MC-500 cement) were added to the cements (dosage 10 ml/kg dry weight cement). This results in the particle size shown in Figure 12.13.

It can be seen that:

– Type A and Type C cements are affected by the dispersants, but coarse particles are not greatly affected. NS-200 is more effective than DARACEM.

– MC-500 microfine cement is affected over the whole particle size range, with the 'coarse' particles also being broken up by the NS-200 dispersant. This results in the dispersed microfine cement being very much finer than the conventional cements. DARACEM does not break up the MC-500 cement.

12.2.4.2 *Viscosity*

Cement grouts behave as Bingham fluids, with a yield point stress which must be exceeded before any flow can occur. This has been discussed by Deere & Lombardi (1985) and Lombardi (1985) who use the term 'cohesion' to describe the yield point. The viscosity and yield point are dependent on water cement ratio.

Cement grouts tend to settle out of suspension during the grouting operation (with the water 'bleeding' out) and it was considered that the water cement ratio at mixing might not be the most critical feature. To gauge this, yield point stress values were determined for Type A, Type C and MC-500 cements at mixing water cement ratios from 3 to 1 to 0.5 to 1, and then also on the grout after allowing 'bleeding' for 15, 30 and 60 minutes. The results are shown in Figure 12.14. The tests were carried out using the shear vane technique described by Nguyen & Bogar (1983, 1985). There is a scatter in experimental results but it can be seen that the yield point stress increases significantly with time as the cement settles.

12.2.4.3 *Estimation of fracture opening from water pressure tests*

The estimation of the opening of fractures in rock in which a water pressure test has been carried out is a complex matter which depends on many factors, some of which cannot be determined in practice. These include the spacing, continuity, degree of intersection, aperture, roughness and inclination of the fracture systems.

The approach usually adopted, is to first assess an equivalent hydraulic conductivity, or 'permeability' k from the water pressure test. This usually uses formulae which assume that the rock behaves as a homogeneous isotropic medium with the joints/fractures sufficiently closely spaced and intersected that the rock behaves as a continuum rather than as a two phase very low permeability substance with discrete fractures. Flow is assumed to be radial from the borehole to a distant 'sink.' Such an approach was used by Snow (1968), Vaughan (1963) and Hoek & Bray (1981). More recent work has attempted to model the flow in rock more correctly (e.g. Witherspoon 1986) but for practical purposes the equivalent permeability approach is the only viable option for civil engineering works. For this study the authors have used the formula given in Hoek & Bray (1981):

$$k_E = q \log_e \left[2m \left(\frac{L}{D}\right) \right] / 2\pi L H_c$$

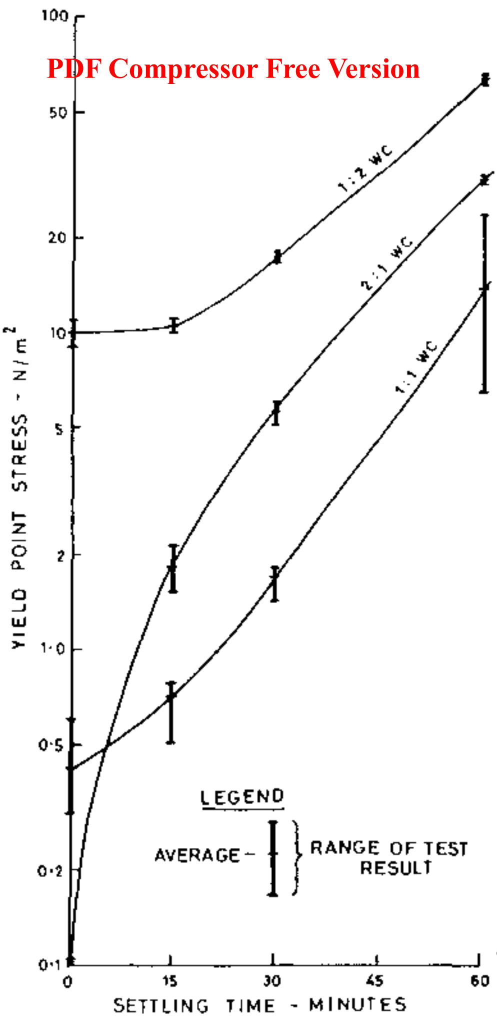


Figure 12.14. Effect of water cement ratio and settling time on yield point stress of cement grout.

where k_E = equivalent permeability of the rock (m/sec)

q = flow rate in water pressure test (m^3/sec)

L = length of test section (m)

H_c = nett pressure head at the test section (m)

D = diameter of borehole being tested (m)

m = factor relating to the ratio of permeability parallel and normal to the borehole, i.e. $m = (k/k_p)^{1/2}$ (where k = permeability normal to the borehole; k_p = permeability parallel to the borehole).

Flow of water (or grout) from a borehole occurs along the fractures intersected by the borehole. In a well planned investigation or grouting program the holes will be oriented to provide the best intersection of the major fracture system or fracture systems. In many practical applications one fracture system will dominate behaviour and hence a simplifying assumption that flow from the borehole will occur along parallel joints is reasonable. Several investigators have carried out experiments to determine the flow characteristics on such a fracture system, including Louis (1969) who indicated that for laminar flow in smooth, clean fractures

$$k_E = \frac{2a^3g}{3bv}$$

and for laminar flow in rough, clean fractures

$$k_E = \frac{2a^3g}{3bv\left(1 + 8.8R^{1.5}\right)}$$

where k_E = equivalent permeability of the rock (m/sec)
a = half aperture width of the fracture (m)
b = joint spacing (m)
g = gravitational constant (9.81 m/sec)
v = kinematic viscosity of water
R = relative roughness of the fracture = (S/4a) ; where S = surface roughness in m.

Sato et al. (1984) carried out further experiments which resulted in similar equations, although no allowance was made for relative roughness. Witherspoon (1986) indicates that experiments carried out by Iwai (1976) confirmed the relation $k_E \alpha a^3$ for apertures with a width as small as 0.01 mm (much finer than the practical limit of grouting).

Figures 12.15 and 12.16 have been prepared using the Louis (1969) formulae.

It can be seen that for rough fractures the fracture spacing has an important effect on the fracture opening resulting in a given Lugeon value, e.g. 10 Lugeons represents an opening of about 0.2 mm for 1 fracture/metre but only about 0.1 mm for 4 fractures/m for m 'rough' fractures.

Note that for a particular rock, the relative roughness will vary for different fracture widths assuming that the surface roughness remains the same. Hence a joint with a surface roughness of 0.05 mm (silt size particles) will have a relative roughness of 0.1 'smooth' for a fracture width of 0.25 mm and 0.5 'rough' for a fracture width of 0.1 mm.

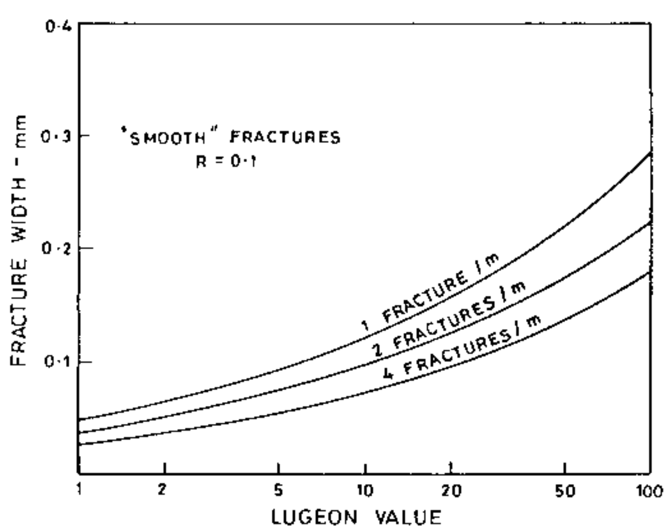


Figure 12.15. Fracture opening versus Lugeon value using Louis (1969).

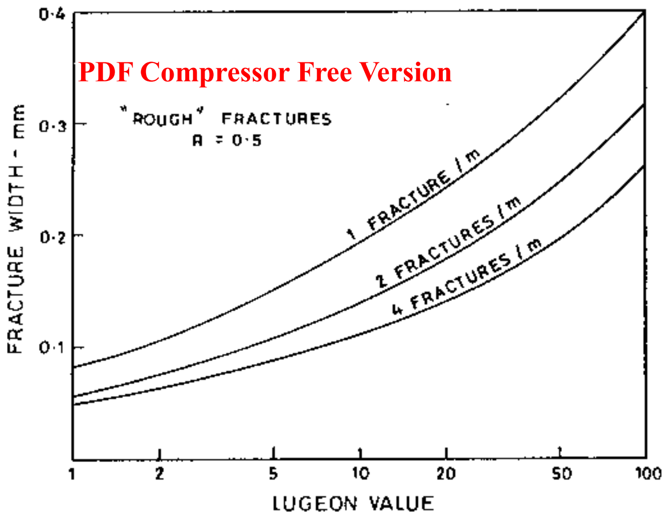


Figure 12.16. Fracture opening versus Lugeon value using Louis (1969).

12.2.4.4 *Penetrability of cement grout*

There are two issues to be considered:

– Are the fractures sufficiently wide to allow grout to enter?

– Having entered, how far will the grout penetrate?

There are different opinions on the width of fracture into which cement grout will penetrate. Snow (1968) quotes Kennedy (1958) as indicating that grout will not penetrate a fracture narrower than 3 times the maximum particle size of the cement. This is also the criterion quoted by Mitchell (1970) and supported by Karol (1985). Houlsby in Houlsby (1982b), and in Water Resources Commission (1981), states that grouting is difficult with a fracture width of less than 0.5mm and gives a lower limit of 0.2 mm. This opinion was confirmed in Houlsby (1986). Littlejohn (1982, 1985) suggests that fractures narrower than 0.16 mm or narrower than $5D_{100}$ cannot be grouted with cement grout. In principle it is reasonable to assume that the ability of cement grout to penetrate a fracture will be dependent on the actual particle sizes of the grout in suspension. This may be considered in relation to the maximum particle size, D_{100}, or by analogy with design of well screens and filters, to the D_{85} size of the grout.

When considered in conjunction with the relation between Lugeon value and fracture width in Figures 12.15, 12.16 and the experience of Littlejohn (1982), it seems reasonable to adopt the Kennedy (1958), Mitchell (1970) and Karol (1985) approach that the limiting fracture width is $3D_{100}$. For the cements tested the limiting widths would be as shown in Table 12.4.

Houlsby's (1982b, 1986) experience that fracture less than 0.5 mm wide are difficult to grout, and less than 0.25 mm impossible to grout is inconsistent with the Lugeon values which can be determined in Figures 12.15 and 12.16 for those fracture openings and the criterion that limiting fracture width = $3D_{100}$.

Some laboratory testing done on cracks of varying thickness under the author's supervision (Tjandrajana 1989) has confirmed that the limiting fracture widths above are reasonable. However, there is some reduction of grout flow with time, and a tendency for a 'filter-cake' of coarser particles to form across the fracture opening, leading to a reduction in cement content of the grout actually penetrating into the fracture to below that being introduced into the grout hole.

Table 12.4. Maximum groutable fracture openings.

Type A	0.18 mm
Type C	0.15 mm
MC-500	0.08 mm

Table 12.5. Estimated minimum Lugeon values indicative of rock which will accept cement grout.

Cement	Minimum Lugeon value which can be grouted		
	1 fracture/m	2 fractures/m	4 fractures/m
Type A	8	16	32
Type C	5	10	20
MC-500	3	5	10
Type A with dispersant	8	16	32
Type C with dispersant	5	10	20
MC-500	1	2	4

Notes:

1. Fractures are assumed 'rough';
2. Fractures assumed to be the same width;
3. One Lugeon is a flow of 1 litre/minute/metre of borehole under a pressure of 1000 kPa. In a 75 mm dia borehole it is approx. 1.3×10^{-5} cm/sec equivalent permeability.

An estimate of the minimum groutable joint aperture, using type 'A' cement has been made from an analysis of site investigation data and grouting records for Plashett Dam (Hawkins 1988). This has shown that very small grout takes occur where water tests are below 6 Lugeons. A significant increase in grout take occurs between 6 and 8 Lugeons. The 'probable maximum aperture' for these values has been inferred from the Lugeon values and joint spacing and found to lie within the range of 0.13 to 0.19 mm which is broadly consistent with those quoted above for Type 'A' cement.

If the adopted figures in Table 12.4 are combined with Figures 12.15 and 12.16 the minimum Lugeon values which can be grouted are as shown in Table 12.5.

Table 12.5 indicates that rock masses giving some quite high Lugeon values cannot be grouted, which may seem contrary to experience. However, the larger values (15 to 30 Lugeons) occur when there are two or more open fractures (joints) per metre. This condition may not be met often, even when there are three or more fractures per metre, as many may not be open. The possibility also exists that when long grouting times are used, settling of the grout occurs in the borehole, with only water with very low (fine) cement contents being left in the upper parts of the hole, allowing only small 'grout' takes to occur in these parts of the hole. Some experiments carried out by Tjandrajana (1989) show this is not a problem for 2:1 water cement ratio, but the upper 1.5 m of the hole is left largely with water after one hour for 5 to 1 water cement ratio. The addition of plasticizers worsens the settling, because the cement particles act alone and actually settle more quickly.

12.2.4.5 *Distance grout will penetrate*

If the fractures are sufficiently open to allow penetration of the grout, the distance to which the grout will penetrate is dependent on the fracture width, grout pressure and viscosity and the time taken in grouting. If grouting continues for sufficient time, the limit of penetration is determined by the yield point stress. Lombardi (1985) showed that:

$$R_{max} = \frac{P_{max}\, a}{C}$$

where R_{max} = maximum radius of penetration (m)
a = half width of the fracture (m)
C = yield point stress (kPa)
P_{max} = grouting pressure (kPa).

Lombardi (1985) also presents a method for estimating the effect of grouting time. A feel for this can be obtained from the data in Deere & Lombardi (1985), which indicates that for medium to thick grouts approximately 75% of maximum penetration will occur in the first hour.

In Figure 12.17 penetration distances have been calculated assuming the yield point stresses in Figure 12.14 and assuming the radius of penetration is 0.75 times that from Lombardi's equation. Bleed times from 5 to 35 minutes have been used.

It can be seen that the radii of penetration using the yield point stresses from the grout which has been allowed to bleed for 15 and 35 minutes are more consistent with actual grouting practice, where it is common to have to grout secondary and tertiary holes down to 3 m, or even 1.5 m to achieve 'closure,' or in cases of closely fractured rock, any reduction in pre-grouting Lugeon value.

Examples of this behaviour are given in Table 12.6, the data for which is taken from Houlsby (1985).

It is recognised that there would have been a scatter of permeability data within each stage of grouting, but on average:

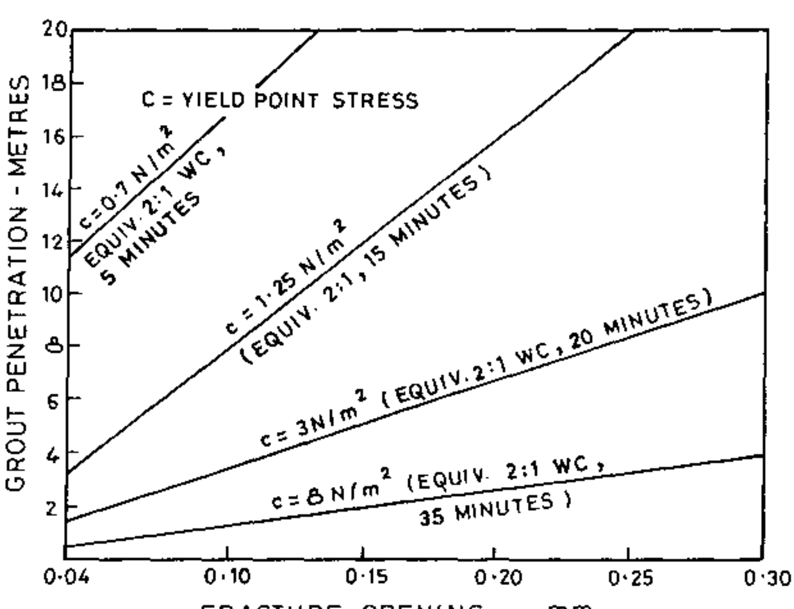


Figure 12.17. Grout penetration estimated from yield point stress and fracture opening.

Table 12.6. Reduction in permeability achieved by grouting (adapted from Houlsby 1985).

Dam	Jointing and rock type	Average permeability before grouting – Lugeons				
		24 m	12 m	6 m	3 m	1.5 m (1)
Copeton	Wide continuous in granite	21	10	9	8	6
Toonumbar	Medium sandstone, conglomerate and shale		38	30	25	10
Glenlyon	Medium-close claystone, calcite and quartz veins		48	42	37	15
Chaffey	Very close, Jasper, siltstone and chert		19	15	18	24

(1) Grout hole spacing

Table 12.7. Approximate penetration from the borehole of grout in fractures.

Lugeon value	Fracture spacing		
	1 m	0.50 m	0.25 m
100	20 m	12 m	4 m
50	12 m	3 m	2 m
20	3 m	1.5 m	1 m
10	2 m	1 m	NP
5	1 m	NP	NP
1	NP	NP	

– At Copeton Dam, grout penetration in the continuous open joints was greater than 12 m, but little benefit was gained after the primary holes were grouted.

– At Toonumbar and Glenlyon little reduction in permeability was achieved until the holes at 3 m spacing had been grouted, indicating a penetration of 1.5 to 2 m in the medium-close jointed rock.

– At Chaffey Dam, no benefit was achieved even with the holes at 3 m spacing, indicating grout penetration was less than 1.5 m in the very closely jointed rock.

The authors have also experienced other conditions which would lead to a reduction in water content of grout, and an increase in yield point stress. These are where porous (often weathered) rock above the water table is being grouted. The water is sucked out of the grout into the dry weathered rock substance or fine fractures.

Comparison of Figure 12.17 and Table 12.6 shows that, apart from Copeton Dam (where the values are consistent with the '15 minute C' value), the actual penetrations are similar to those which can be assessed from the 30 minute yield point stress values.

It is considered reasonable to conclude that the penetration of Type A and C cement grouts will be of the order of those shown in Table 12.7. In the table NP indicates the grout will not penetrate the fractures, so grouting will be ineffective.

12.2.4.6 *The effectiveness of a grout curtain reducing seepage*

Figure 12.18 shows a simplified typical embankment and foundation section with a low permeability core, high permeability rockfill shoulders, and a foundation consisting of an upper layer of permeable rock overlying 'impermeable' rock.

Figure 12.19 presents the results of an approximate seepage analysis to show the effective-

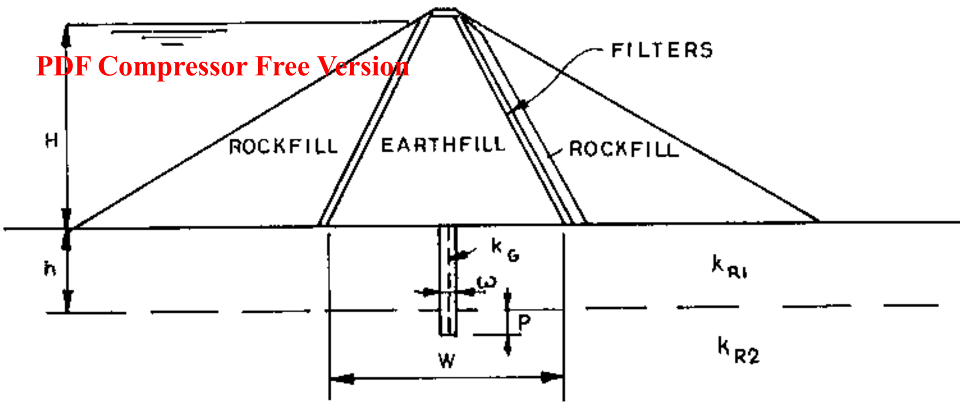


Figure 12.18. Generalised seepage model.

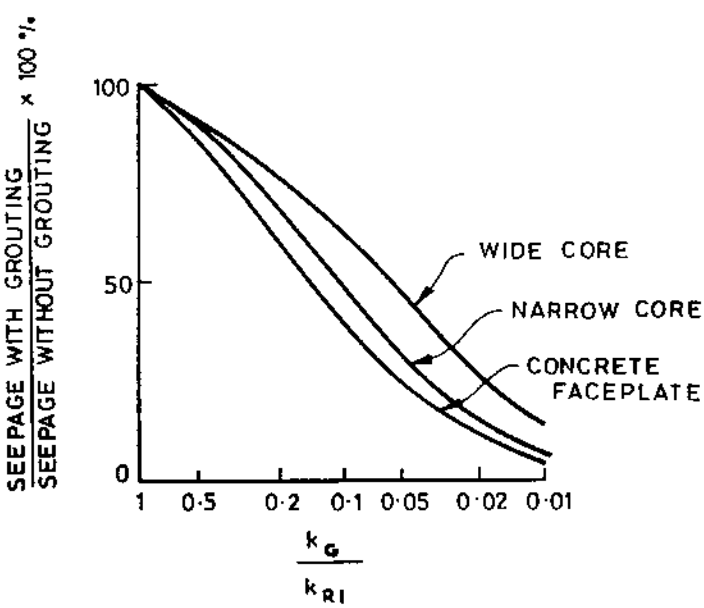


Figure 12.19. Effectiveness of grouting in reducing seepage.

ness of a grout curtain in reducing the amount of seepage beneath the embankment. The analysis has been based on

H = 60 m all cases
W= 60 m for wide core
w = 6 m all cases
W= 20 m for narrow core
h = 20 m all cases
W= 6 m for concrete face plate

Unless the grout curtain has a permeability at least 10 times lower than the ungrouted rock the reduction in seepage is less than 50%. So, for 20 Lugeon rock, the grout curtain must be less than 2 Lugeons to achieve 50% reduction in seepage. As pointed out above, it is doubtful whether such a low permeability is achievable with cement grouting because the cement will not penetrate rock with such low Lugeon value.

For high permeability rock (say 100 Lugeon) with wide fracture spacing, the effective width at the cutoff will be larger (say 20 m) and the permeability could be reduced to that of the secondary fracture system. If this was say 5 Lugeons, then the seepage would be reduced to about 20% of the ungrouted value. If the permeability of the secondary fractures was, say, 2 Lugeons, the seepage would be reduced to 5 to 10% of the ungrouted value.

12.2.5 *Recommended closure criteria*

From the review of criteria used by others, and the discussion in Section 12.2.4 it is recommended that closure criteria should be based on:

– Lugeon value prior to grouting the hole,
– grout take,
– the nature of the dam, its foundation and what is being stored in the dam.

These factors should be considered together to make decisions about whether further grouting is required.

It is not yet possible (and may never be) to quantify all these effects, mainly because of the complexity of flow in fractured rock, and the time dependent nature of cement grout properties. It is also not possible to make rigid rules – each case should be considered on its own merits. As a guide, it is suggested that for grouting of the foundations of earth and earth and rockfill dams with Type C portland cement the guidelines given in Table 12.8 be adopted.

'Erodible' foundations would include extremely to highly weathered rock, and rock with clay filled joints which might erode under seepage flows.

For grouting with Type A portland cement, Lugeon values should be increased by say 20% to account for the coarser nature of the grout particles. For microfine cement grouting at values half those quoted would be reasonable from a grout penetrability viewpoint, but may not be justified by the benefit gained.

The overriding philosophy in these recommendations is that it is not possible to stop seepage by grouting, only to reduce it, and cement grout can only significantly reduce seepage when it can penetrate the fractures i.e. in relatively widely spaced open joints in moderate to high Lugeon value rock. The objective of the grouting operation should be to locate and and fill these large fractures and to thereby avoid high and concentrated seepage flows.

Different criteria have been given for 'erodible' foundations, with some misgivings. It is better to acknowledge that if foundations are erodible grouting will not prevent erosion, only reduce erosion potential. With or without grouting, filters should be provided under the down-

Table 12.8. Guidelines for deciding on limits of effective grouting with Type C portland cement.

Erodibility of foundation	No further grouting needed when Water test value prior to grouting (Lugeon)	or < 20% reduction in Lugeon value or grout take from previous stage (1) (Lugeon)	or All grout takes (kg cement/m)	or Grout hole spacing (m)
Low/non	< 10	< 20	< 25	< 1.5 m
High	< 7	< 15	< 25	< 1.5 m

(1) For rock with joints closer than 0.5 m.

tream part of the dam to allow seepage water to emerge in a controlled manner without erosion of the foundation.

For concrete faced rockfill, gravity concrete and arch dams seepage gradients are higher and there is some argument for using lower Lugeon values. However, the overriding consideration is that the cement will not penetrate fine fractures, so at most, the values quoted should be reduced by about 30%.

The question of whether the water is 'precious' or has a high contaminants content is not really relevant, since grouting to more restrictive Lugeon values as advocated by Houlsby (1986) will not result in significant reduction in seepage. In these cases it will be necessary to collect the seepage water downstream of the dam, either in a catch dam or in seepage collector wells.

A minimum grout hole spacing of 1.5 m is recommended because, if closer spacing is required to achieve a Lugeon value standard, the width of the resulting grout curtain is too narrow to significantly reduce seepage from the curtain formed by the 1.5 m spacing. Clearly, there may be exceptions where large takes are encountered in these holes, but as a general rule grouting with holes at closer than 1.5 m spacing is a waste of time and money.

The authors also are of the opinion that it may be justified to reduce hole spacings below 1.5 m in some deeply weathered and closely fractured saddle dam foundations, where erodibility, and control of seepage pore pressures may be critical for slope stability.

It should be noted that adoption of larger Lugeon value closure criterion at depth, and/or higher grout pressures at depth may lead to secondary and tertiary holes not penetrating to the same depth as the primary holes.

12.2.6 *The depth and lateral extent of grouting*

So far as is practicable grout holes should be taken to the depth at which the Lugeon closure criteria is achieved. In near horizontally layered rock this might be clearly identifiable as a lower permeability zone of rock beneath the valley floor, e.g. siltstone within interbedded siltstone and sandstone, but may be at variable depths around the dam abutments due to the influence of stress relief, weathering and rock types. Figure 12.20 gives an example where the stratigraphy is simple and clearly known.

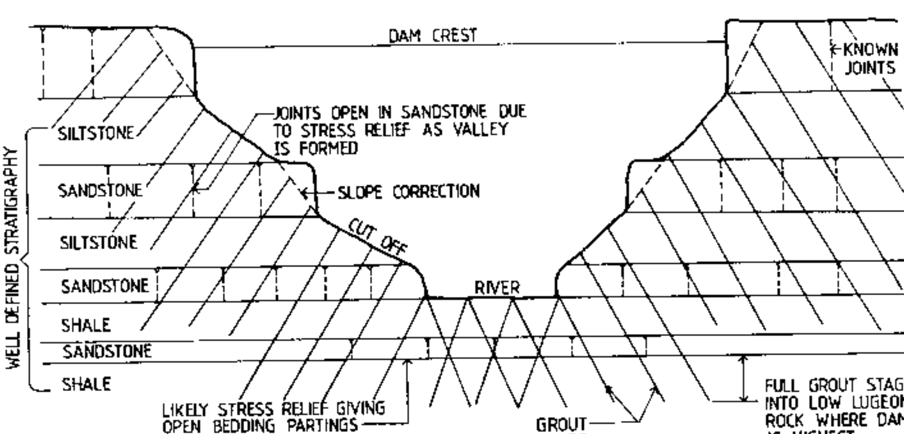


Figure 12.20. Example of grout curtain in a simple sedimentary rock environment.

The depth of higher Lugeon value rock is seldom known with any accuracy, and the grout holes must penetrate to at least one stage below the estimated base level to prove that the necessary conditions have been met.

The use of 'rules of thumb' to determine the depth of grouting is not recommended as there is no logical basis. Some examples of such rules of thumb are given in Ewart (1985) and Houlsby (1977, 1978) (neither of whom promote their use), e.g:

$$\text{Depth} = H/3 + C$$

where H = height of dam in m

C = 8 to 25 m depending on type of foundation, size of dam, importance of leakage

or Depth = height of dam

or Depth = 2/3 height of dam.

If such rules are applied they will either yield holes which are too shallow, resulting in a grout curtain which only partially penetrates the permeable foundation, giving only minor (i.e. typically less than 10%) reduction in seepage due to increased seepage path length, or more commonly, holes which are much deeper than required to reach lower permeability rock, particularly in the river section of the foundation where the dam is highest.

For most dams, grouting will extend up the abutments to where full supply level intersects the base of the permeable zone. However, where the dam abuts a spillway, the grout curtain may be connected into the curtain under the spillway, and/or if the dam abuts a relatively narrow permeable ridge, the grout curtain may be extended into the ridge. Figure 12.21a shows an example of this.

Extension of the grout curtain into the abutments is only likely if the stability of the ridge or abutment is in question, and there is a need to control piezometric pressures, or if the abutment is highly permeable.

In highly permeable rock, careful consideration must be given to the lateral extent of grouting, or seepage may bypass the end of the grout curtain. Figure 12.21b shows the grout curtain for a 20 m high water storage dam in coal measure rocks, which have been intruded by a highly permeable dolerite sill. The grout curtain was successfully constructed with excellent closure, but when the dam was filled the reservoir leaked significantly (water level dropped 25 mm/day) because water was entering and flowing along the dolerite sill, completely bypassing the ends of curtain. This groundwater movement was proven conclusively by piezometers.

12.2.7 *Grout hole position and orientation*

For earthfill and earth and rockfill dams the grout curtain will usually be located in the centre of the cutoff trench, which in turn will usually be located at the centre or upstream of centre of the earthfill zone. Houlsby (1977, 1978) presents an argument for positioning grout curtains upstream of centre of the earthfill zone, to maintain a greater seepage pressure head in the earth core than in the foundation. This concept has merit in principle, but in practice flownets within the earthfill and foundation are influenced by different permeabilities and positioning the curtain upstream of the centre of the core may not achieve the desired objective.

For concrete faced rockfill dams the grout curtain will be positioned in the plinth, usually, but not always, with a line of consolidation grout holes upstream and downstream of the curtain to give better grouting protection close to the surface near the plinth, where gradients are high.

For concrete gravity dams (including spillways) the grout curtain should be positioned as close to the upstream face as possible so that the drainage holes can be positioned downstream

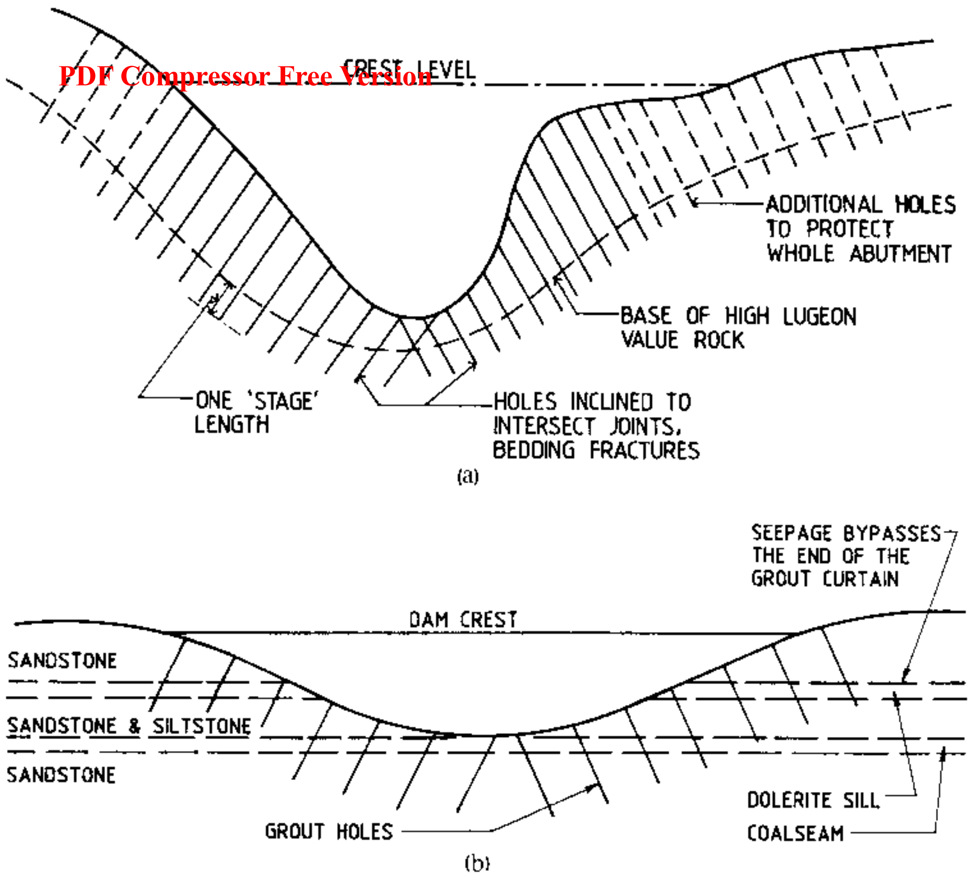


Figure 12.21. a) Example of grout curtain extended beyond dam crest. b) Example where seepage bypassed the end of the grout curtain.

of the grout curtain but still close to the upstream face to reduce uplift pressures as much as possible. It should be noted that for these structures the drain holes are critical in reducing uplift pressures and are more important than grouting. Grouting alone cannot be relied upon to reduce uplift pressures.

Grout holes should always be oriented to intersect the major fracture sets, particularly those which are near parallel to the river. In most cases this will mean that the holes are inclined from the vertical as shown in Figures 12.20 and 12.21. They may also need to be inclined upstream to intersect joints perpendicular to the river.

12.3 SOME PRACTICAL ASPECTS OF GROUTING WITH CEMENT

12.3.1 *Grout holes*

It has been accepted practice in Australia and many other countries for many years to use wet percussion drilling for grout holes. Holes are a 30 mm minimum diameter; usually 50 mm diameter and seldom larger than 60 mm.

Percussion drilling is the quickest and cheapest type of drilling and is satisfactory except where the nature of the rock is such as to create a sludge or stiff clayey 'plug' which blocks the fractures in the rock. In such rock rotary drilling using roller bits, or plug diamond bits, may be used (Houlsby 1977, 1978). It is argued that rotary or diamond drilling will be less likely to block the fractures. If holes are washed carefully after drilling it is doubted whether the drilling method really has much effect.

Deere (1982) and US Corp of Engineers (1984) indicate that US practice is changing from rotary drilling, to the acceptance of percussion drilling, including down-the-hole hammers, where rock conditions permit, i.e. where 'slimes' are not found or where the bit does not 'plug.'

Ewart (1985) indicates that rotary drilling may be preferred where packers are to be used as this will usually give a smoother hole.

Bruce (1982) indicates that in British dams, holes are usually drilled rotary percussion with water flush and that they are typically 50 mm diameter. The Swiss National Committee on

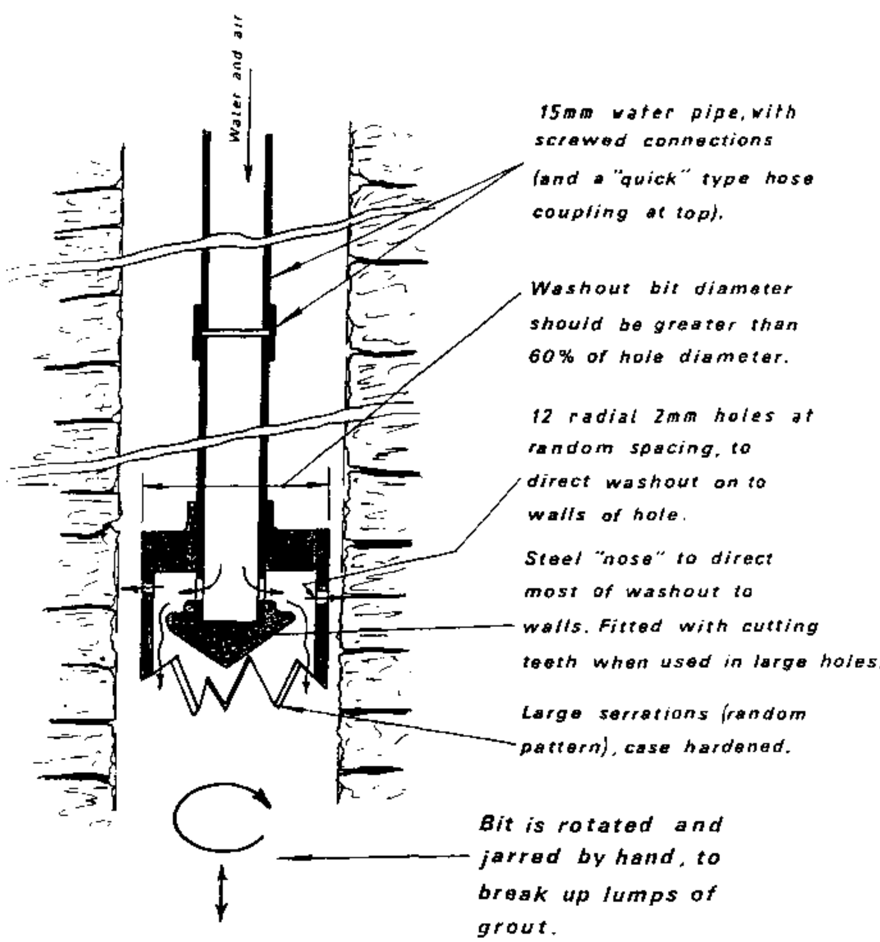


Figure 12.22. Grout hole washing bit (Water Resources Commission 1981).

Large Dams (1985) indicates that diamond coring was used for grout holes on earlier dams because it was the only practical method available for the 100 to 150 m deep holes. It was also considered that there was less deviation of the hole (from straight) and fewer cuttings to clog fractures. However, later practice was to use of percussion drilling, washed thoroughly with water.

Washing of the grout hole before grouting is essential to remove cuttings which have clogged the fractures. This is done by lowering a specially constructed washout bit which directs water under pressure against the sides of the hole. Figure 12.22 shows an example of such a bit. This bit may also be used to flush grout from a hole after grouting.

12.3.2 *Standpipes*

Standpipes are used for most grouting operations where packers are not used. The standpipe consists of a threaded galvanised pipe just larger than the drill size, set approximately 0.6 m into the rock as shown in Figure 12.23.

Standpipes will always be required where the rock near the surface is fractured or weak, precluding use of a packer. Standpipes have the advantages of preventing clogging of the hole by debris from adjacent areas, and providing easy hook-up for grouting. Note that the part of the standpipe projecting from the surface has to be removed before placing earthfill.

12.3.3 *Grout caps*

Concrete grout caps (Fig. 12.24) are required when grouting closely fractured or low strength rock in the situation where a standpipe cannot be sealed into the foundation and/or leakage of grout to the surface will be excessive. The grout cap also provides a better cutoff through the upper part of the foundation than is practicable with grouting from standpipes.

Grout caps should be excavated into the foundation rock without explosives and if practi-

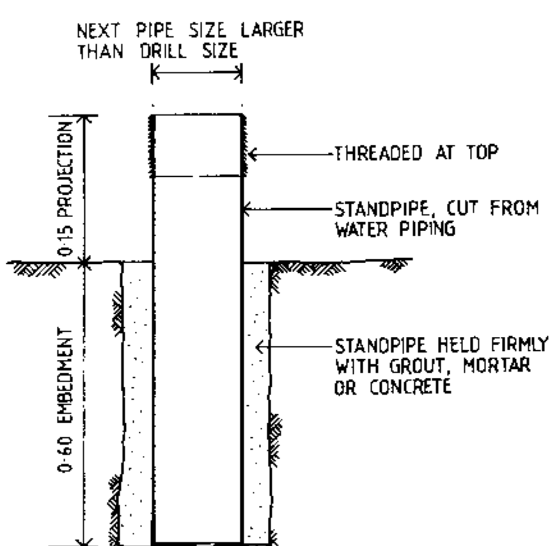


Figure 12.23. Grout standpipe.

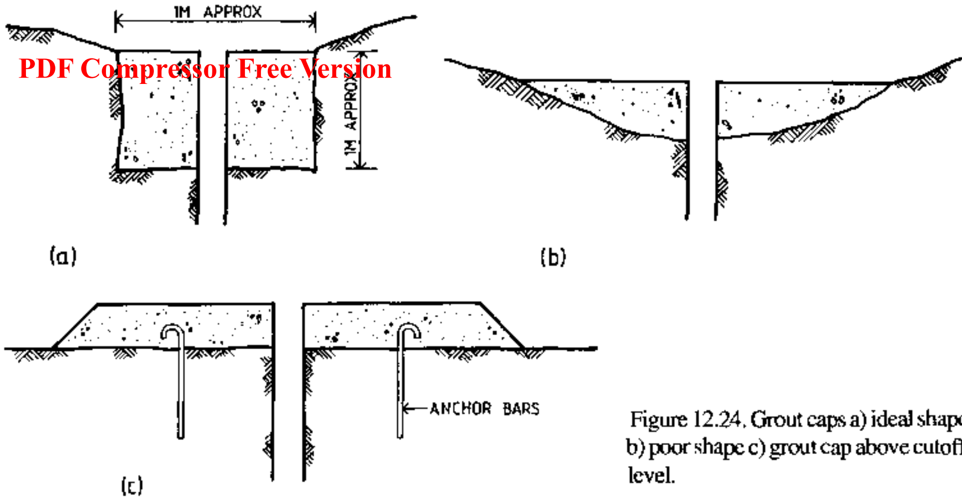


Figure 12.24. Grout caps a) ideal shape b) poor shape c) grout cap above cutoff level.

cable should be of square or rectangular section as shown in Figure 12.24a. This gives good resistance to lifting of the cap under the pressure of grouting.

The wider/shallow shape shown in Figure 12.24b and c is prone to displacement during grouting and may need to be anchored into the foundation with steel dowel bars grouted say 2 m into the rock.

Some organisations, e.g. Rural Water Commission of Victoria, have favoured construction of a grout cap in all foundations, constructed above the cutoff level as shown in Figure 12.24c. This arrangement requires anchor dowels in all cases. The arrangement is justified on the basis of allowing grouting of all the rock, i.e. right to the surface, and increasing the resistance to erosion along the cutoff surface by providing a more tortuous path for water, but has the disadvantage that it interferes with compaction of earthfill in the cutoff trench. Such grout caps are not recommended by the authors.

In closely fractured rock there is often some advantage in applying a layer of slush concrete or shotcrete to the cutoff surface before grouting. This allows use of higher grout pressures, prevents excessive leakage to the surface, and generally facilitates grouting. It also prevents damage to the cutoff surface by construction equipment during the grouting operation.

12.3.4 *Grout mixers, agitator pumps, and other equipment*

Houlsby (1977, 1978) and in Water Resources Commission (1981) strongly advocates the use of high speed, high shear, 'colloidal' mixers. This opinion is supported by US Corps of Engineers (1984), Deere (1982), Gourlay & Carson (1982) and Bruce (1982). Figure 12.25 taken from Water Resources Commission (1981) shows some suitable mixers. The most commonly used mixers are the 'colcrete' or 'cemix' type, which operate at 2000 and 1500 rpm respectively. The very high speed facilitates mixing by separating cement particles from each other and wetting the surface of each particle. It is claimed (Water Resources Commission 1981) that mixing with such high speed mixers 'produces a grout which more closely resembles a colloidal solution rather than a mechanical suspension.' While relatively speaking the grout may settle more slowly, one should not lose sight of the fact that it still is a suspension of cement particles which do settle with time.

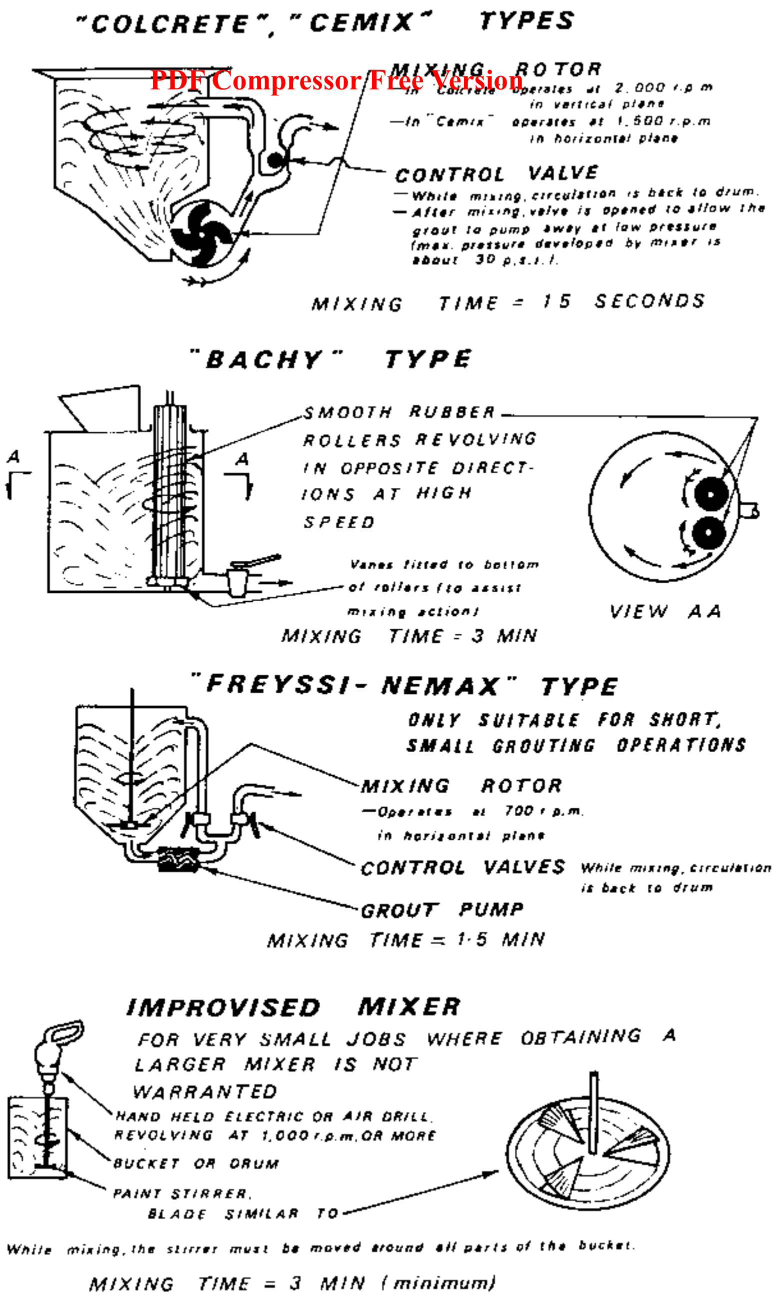


Figure 12.25. Grout mixers (Houlsby 1977, 1978, Water Resources Commission 1981).

Having mixed the grout it is usually transferred to an agitator from which it is pumped to the grout holes. The agitator is relatively slow speed and designed to prevent the cement particles from settling. Figure 12.26 (from Water Resources Commission 1981) shows the desired arrangement.

The quantity of grout injected is measured at the agitator.

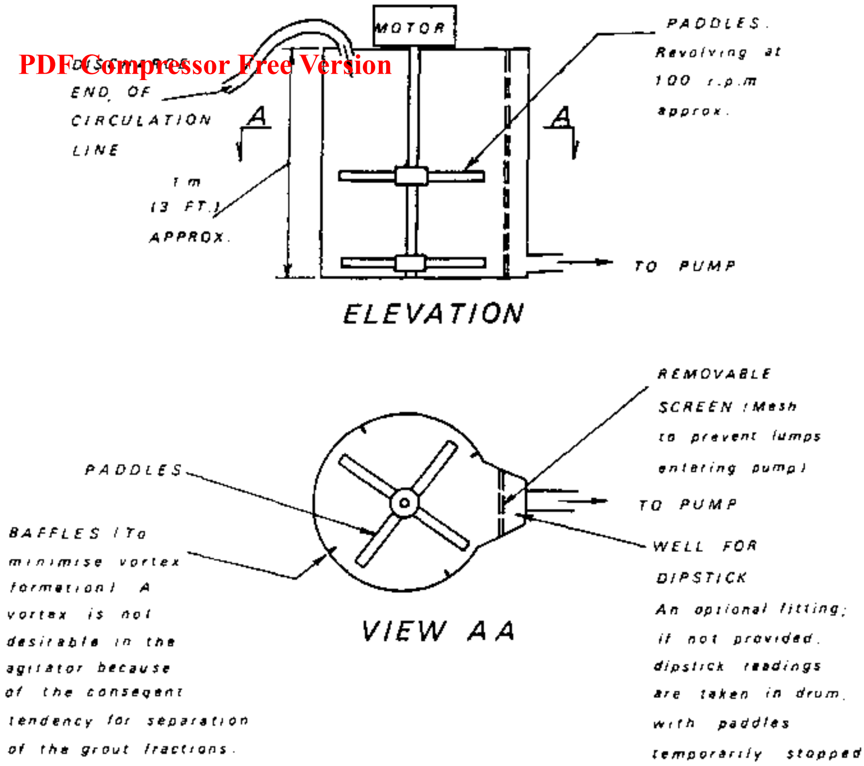


Figure 12.26. Grout agitators (Houlsby 1977, 1978, Water Resources Commission 1981).

Grout is pumped from the agitator using grout pumps. Houlsby (1977, 1978) and Deere (1982) indicate that in Australian and USA practice it is common to use 'mono' (or 'moyno') helical screw pumps because they provide a constant pressure, are rugged and readily maintained. A grout bypass valve at the hole is required as shown in Figure 12.27.

Gourlay & Carson (1982) and Bruce (1982) point out that in British and European practice there is a preference for ram type pumps, as it is believed the pulsating pressure assists in preventing clogging of the fracture opening by coarser grout particles. Deere (1982) also indicates a preference for this type of arrangement, largely because it obviates the need for a bleeder valve at the hole and/or recirculation line. As pointed out by Gourlay and Carson, there is a lack of hard evidence to support either preference and one should be willing to use the equipment readily available.

Houlsby (1977, 1978) advocates the use of a recirculation line of the smallest diameter practicable to keep grout velocities high and avoid blockage. He suggests the maximum size is 25 mm diameter. Deere (1982) suggests that the recirculation line is used in US practice because of the then traditional use of high water cement ratio grouts which were 'unstable' (i.e. settled out) and the requirement for a bypass valve to control injection pressure.

Details of bleeder valves and bypass, flowmeters, pressure gauges and standpipe fittings are given in Houlsby (1977, 1978), Water Resources Commission (1981) and Gourlay & Carson (1982).

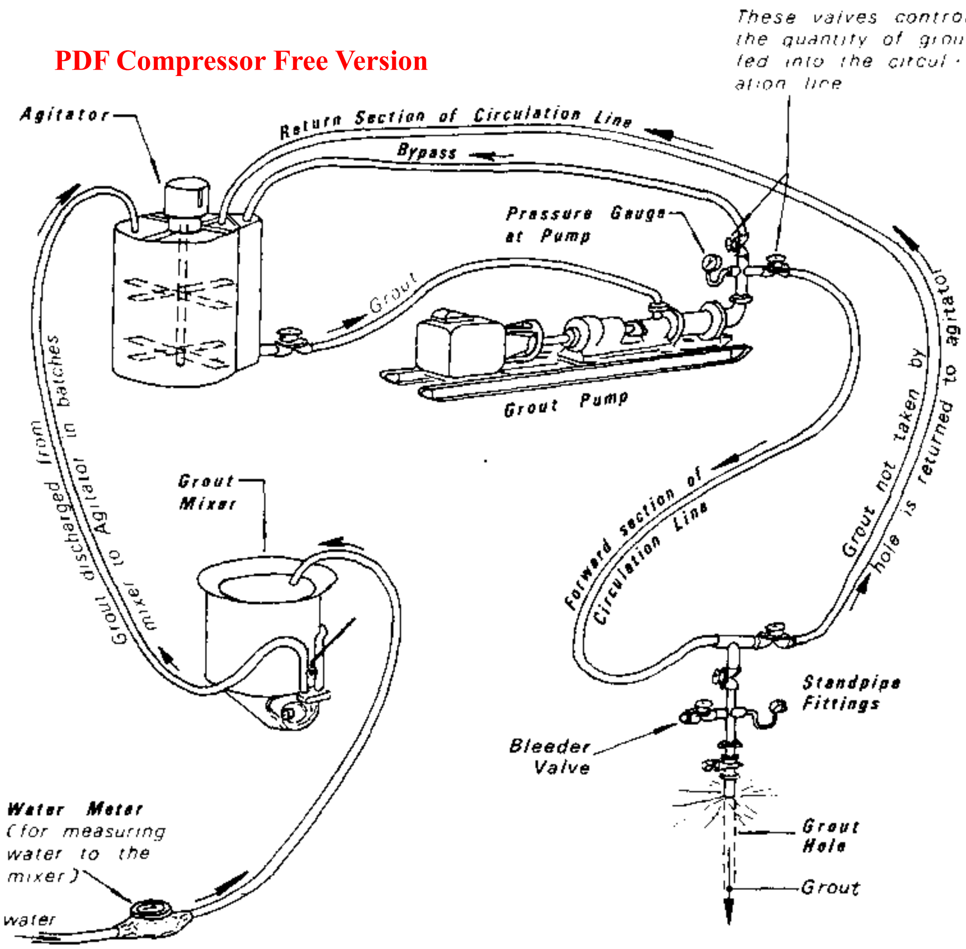


Figure 12.27. Typical arrangement of grouting equipment (Houlsby 1977, 1978, Water Resources Commission 1981).

Packers for use in grout holes are either mechanically operated from the surface by a screw device, or inflatable, expanded against the side of the hole by compressed air or water. The length of 'seal' formed by mechanical packers is often only 0.3 m, compared to 0.5 to 1.5 m for the inflatable type, so leakage past the packer is more likely with the former. Mechanical packers would usually only be adopted if rock conditions resulted in puncturing of inflatable packers.

12.3.5 *Water cement ratios*

Cement grout mixes are usually designated by water cement (WC) ratios, with mixes ranging from 6:1 WC (by volume) to 0.6:1. Most grouting uses mixes of 2:1 WC ratio or less, as it is well documented that higher WC ratios yield unstable mixes (i.e. the particles settle quickly) and the grout is of poor durability (e.g. see Houlsby 1985, Deere 1982 and Deere & Lombardi 1985).

Use of volumetric WC ratios has been traditional in Australian and USA practice because a bag of cement was taken to be one cubic foot, and as such was an easy measure in the field. The

Table 12.9. Relationships of different mix proportions of cement and water in grouts.

Water: Cement by volume	Water: Cement by weight	Cement: Water by weight	
6:1	4:1	1:4	(0.25)
4:1	2.67:1	1:2.67	(0.37)
3:1	2:1	1:2	(0.5)
2:1	1.33:1	1:1.33	(0.75)
1.5:1	1:1	1:1	(1.00)
1:1	0.67:1	1:0.67	(1.50)

relationship between volumetric and weight based ratio is approximate because it depends on bulking of the cement. Table 12.9 gives approximate relationships given by Deere (1982).

Deere (1982) advocates the use of unit weight of the cement grout slurry (measured by a 'mud balance') and a marsh funnel to measure viscosity as part of the control of grout quality in the field. There is some merit in this suggestion as grout penetrability is dependent on viscosity. However, a marsh funnel measures apparent viscosity, and as discussed in Section 12.2.4 the yield point stress may be more important when considering grout penetration near refusal.

The selection of water-cement ratio is to an extent related to the 'stability' of the grout mix. As outlined by Deere (1982) stability is measured by a sedimentation test, in which a litre of grout is placed in a standard 1000 ml graduated cylinder. At the end of 2 hours the volume of clean liquid that has formed at the top of the cylinder due to sedimentation is noted. This volume, expressed as a percentage of the total volume, gives the percentage 'bleeding' or sedimentation.

Figure 12.28 shows results of such tests on grout with different WC ratios, and proportions of bentonite added. Grout with a high WC ratio is 'less stable' and bleeding or sedimentation is large.

Addition of small percentages of properly hydrated bentonite improves the stability. Deere (1982) indicates that in European practice, a stable mix is one which has less than 5% sedimentation. Addition of 2% bentonite results in a small increase in viscosity of the grout but improves stability markedly (Fig. 12.28). There would seem to be some argument for use of bentonite for WC ratios greater than 1:1 (by volume).

WC ratios which should be used for grouting are discussed in some detail by Houlsby (1977, 1978), Water Resources Commission (1981), Deere (1982), Deere & Lombardi (1985), Bruce (1982) and Bozovic (1985)

Houlsby (1977, 1978, 1985) recommends use of WC ratio (by volume) of not more than 3:1 and indicates doubts on long term durability if grout with WC ratios greater than 5:1 are used. He recommends use of the thickest possible mix at all times and suggests the following:

Starting Mix
- 2:1, most sites
- 3:1, for rock < 5 Lugeons
- 1:1, for rock > 30 Lugeons
- 0.8:1, for very high losses
- 4:1, for heavily fractured, dry; or
- 5:1, rock above water table where excess water is absorbed by the dry rock.

Thicken the mix
- to deal with severe leaks
- after 1-1/2 hours on the one mix with continued take (except for 1:1 and thicker mixes)

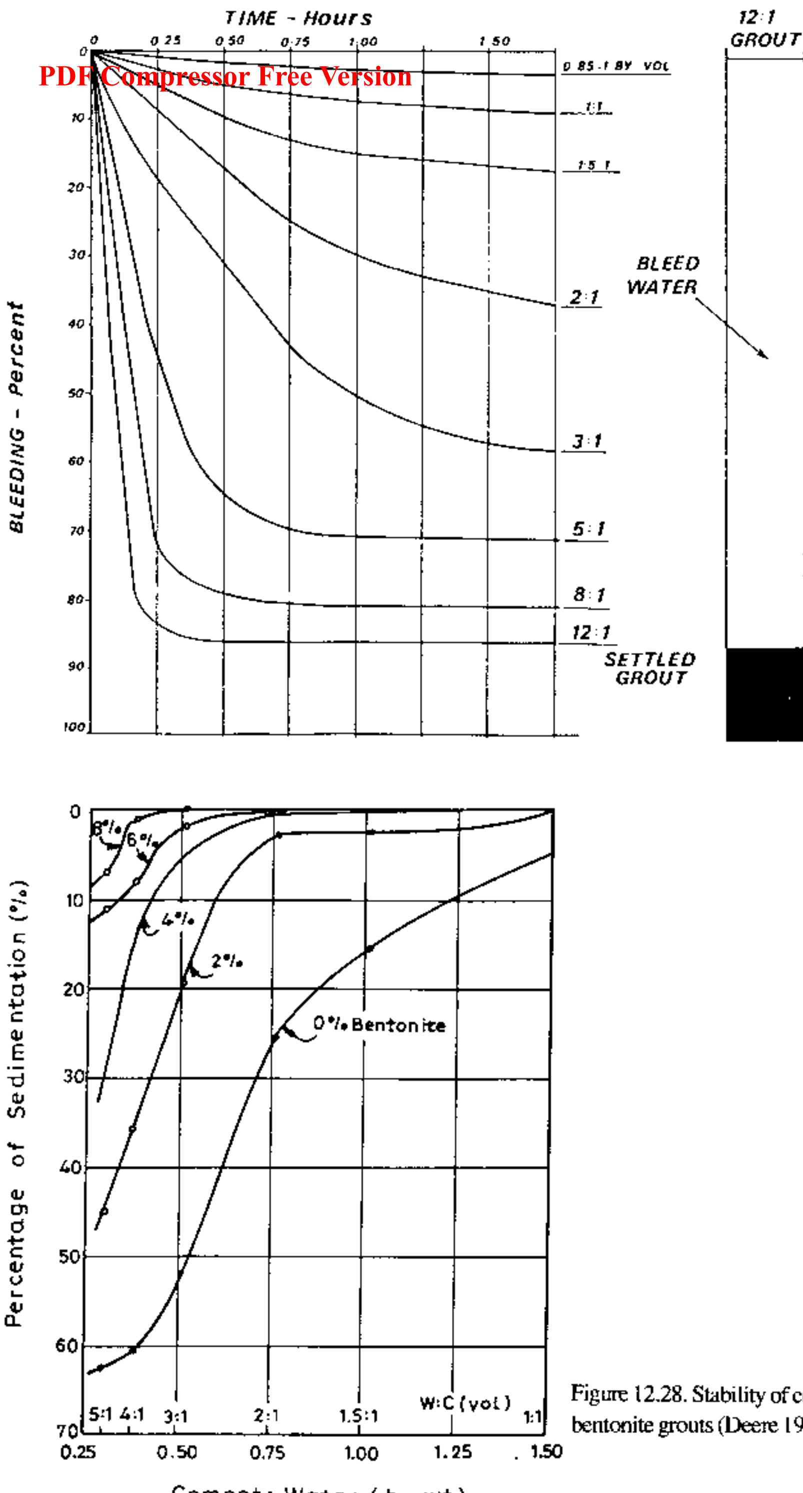


Figure 12.28. Stability of cement and cement-bentonite grouts (Deere 1982, Houlsby 1985).

– if hole is taking grout fast, e.g. > 500 litres in 15 minutes.

He indicates, further, that thickening should be in small increments, e.g. 3:1 to 2:1 and suggests reapplication of grout to the hole if take has exceeded 0.25 litres/cm of hole for WC 2:1 or thinner; or 0.5 litres/cm for WC 1:1 or thicker to fill voids of bleed water. He does not favour the use of bentonite to give stable mixes, preferring to use low WC ratios, and bleeding of the water from the hole

Deere (1982) indicates use of WC ratio (by volume) of around 1.5:1 along with 2% of bentonite to maintain stability. He indicates that the US Corp of Engineers prefer use of grouts around 1:1 WC ratio. In Deere & Lombardi (1985) it is pointed out that successful grouting had been achieved in several major dam projects with 1.5:1 WC ratio (by volume) with between 0.5% and 2% bentonite.

Bruce (1982) indicates use in British dams of WC ratios of 12:1 to 1.5:1 (by volume). Bentonite may be added in some cases.

Bozovic (1985) notes the trend to water cement ratios lower than 5:1 (by volume) but indicates WC ratios as high as 10:1 have been successful provided sufficient pressure is used.

The authors' own preference is to follow Houlsby's approach. It can be seen from the sedimentation figures in Figure 12.28, and the yield point stress values in Figure 12.14, that the higher WC ratio grouts will tend to settle to lower WC ratios. Hence higher viscosity results within the time of grouting, and there appears to be little basis for using higher water contents to improve penetrability. The authors feel there is merit in adding bentonite to mixes with WC ratio greater than 1:1 (by volume) to control sedimentation. There appears to be evidence from major grout projects that it is practical and beneficial to do so.

12.3.6 *Grout pressures*

As pointed out by Deere (1982), Deere & Lombardi (1985) and Lombardi (1985), the maximum penetration distance is proportional to the pressure used for grouting. Hence it is desirable to use as high a pressure as practicable without fracturing the rock.

The pressure which can be applied depends on the rock conditions (degree of fracturing, weathering, in-situ stresses and depth of the water table) and whether grouting is carried out using a packer which is lowered down the hole at each stage (i.e. downhole with packer grouting), or from the surface. The downhole packer method allows progressively higher pressures.

Houlsby (1977, 1978) and Water Resources Commission (1981) presents graphs to allow estimation of maximum pressures at the ground surface. These are based on the assumption that the maximum pressures at the base of the stage being grouted are given by:

$$P_B = \alpha d$$

where P_B = pressure at base of hole in kPa
α = factor depending on rock conditions
≈ 70 for 'sound' rock
≈ 50 for 'average' rock
≈ 25 to 35 for weak rock
d = depth of bottom of stage below ground surface in metres.

This allows for the weight of the overlying rock plus some spanning effect.

Deere (1982) points out that the 'rule of thumb' used in USA (also used in Australia) is that the maximum pressure measured at the surface is 1 psi/foot depth (equivalent to an 'α' value of approx. 24), and that in European practice, pressures up to 100 kPa/m (α = 100) are adopted. He suggests that the lower values should be used in the upper part of a hole where fracturing or 'jacking' can occur (particularly where rock joints or bedding are parallel to the surface) and that the higher values can be used at depth.

The tendency for rock to fracture or 'jack' under grout pressures is reduced by using a relatively low pressure to start grouting, and building up with time. Since much of the pressure in the grout is dissipated in overcoming the viscosity effects in the fracture, this limits the pressure transmitted to outer parts of the grout penetration.

Houlsby in Water Resources Commission (1981) suggests use of a starting pressure of 100 kPa (or less) for 5 minutes, then steadily increasing the pressure over the next 25 minutes until the maximum pressure is reached.

The occurrence of fracturing can be detected by sudden loss of grout pressures at the top of the hole, by increased take, surface leakage, or by monitoring levels of the surface above the rock being grouted. Details of uplift gauges are given in Water Resources Commission (1981).

It is recommended by Houlsby that grouting is to 'refusal,' and that the pressure is maintained for 15 minutes after this to allow time for initial set. Others suggest grouting until take is less than a certain volume in a 15 minute period, e.g. WAWA (1988) specify grouting is to cease when take is less than 30 litres/20 min. at 700 kPa or less; 30 litres/15 min. at 700-1400 kPa; 30 litres/10 min. for pressures greater than 1400 kPa. They also indicate pressures should be maintained until 'set' has occurred.

12.3.7 *Monitoring of grouting program*

It is absolutely essential that detailed records of the grouting operation are kept. This is necessary to allow progressive development of the grouting operation, e.g. decision making on hole depths, whether closure holes are required, whether grout mixes should be changed. In most cases, detailed records will also be needed for payment purposes. Matters recorded should include:

– hole locations, orientation, depths;
– stage depths;
– water pressure test value for each stage prior to grouting, including maximum pressure used;
– grout mix(es);
– grout pressures, takes, at for example, 15 minute intervals and then in summary;
– grouting times;
– leaks, uplift;
– total grout takes for each stage;
– amount of cement in these takes;
– cement takes/unit length of hole.

This data will be collected largely by foremen but an engineering geologist should be involved continuously to interpret the progress of the grouting, relate this to the geotechnical model of the foundation, and make decisions on closure etc. This will invariably involve keeping records on sections along and across the grout centreline with geological data superimposed.

12.3.8 *Water pressure testing*

Before grouting each stage, a water pressure test should be carried out. This is done using a method similar to that outlined in Chapter 7, but using a simplified procedure, e.g. applying only one pressure (100 kPa) for 15 minutes, taking flow quantities at 5, 10 and 15 minutes to estimate the Lugeon value.

12.3.9 *Type of cement*

Type A (AS1315-1982) portland cement is commonly used (Type I, ASTM) or Type C sulphate resistant (Type II) if acid groundwaters warrant, e.g. in some tailings dams. There is some argument for using Type C as it is marginally finer than Type A and should penetrate fine fractures more readily.

Deere (1982) and Bozovic (1985) indicate that on some South American and Japanese projects, special cements have been manufactured with additional grinding as outlined in Section 12.2.4. There are microfine cements available with significantly finer particle size, and enhanced ability to penetrate fine fractures. These are, however, significantly more expensive than conventional cements.

12.3.10 *Prediction of grout takes*

For contractual and cost estimating reasons it is necessary to estimate grout takes, i.e. the volume of grout (or dry weight of cement) which will be absorbed by the foundation during the grouting operation.

This is difficult to do with any degree of accuracy because the penetration of the grout is dependent on fracture aperture, roughness, continuity and interaction with other sets of fractures; and grout viscosity, pressure, duration etc, etc.

As mentioned in Section 12.2 by Bozovic (1985) in his general report of the ICOLD Congress the correlation between grout take and Lugeon value is very weak. He suggested that considering the different rheological properties of grout and water, a correlation cannot physically exist.

Figure 12.29 shows some data from Ewart (1985) which shows the poor correlation. Ewart (1985) presents similar data for Aabach Dam, as do Sims & Rainey (1985) for Gitaru Dam.

The authors agree that if grout take and Lugeon value are compared directly, there is a poor correlation. However, they are of the opinion that if joint spacing is used to estimate fracture openings, and grout penetration is estimated from grout particle size, grout pressure, viscosity and time, it should be possible to obtain a better prediction of grout take. It is not expected that accurate prediction will result, only an improvement in the ability to predict. If this approach is coupled with grouting trials on the dam foundation it should be possible to achieve reasonable accuracy.

That prediction has been a problem is emphasised by the poor correlation between predicted and actual grout takes reported in US Corps of Engineers (1984) as shown on Figure 12.30.

Note that in most cases actual takes were less than estimated, and the ability to estimate does not appear to have improved significantly with time.

It is concluded that the best approach is to gather data from dam sites in similar geological environments, and make initial estimates of take per metre of grout hole on this basis. For any reasonable degree of accuracy trial grouting on a representative part of the dam foundation will be required, with careful monitoring of takes in primary, secondary, tertiary holes etc.

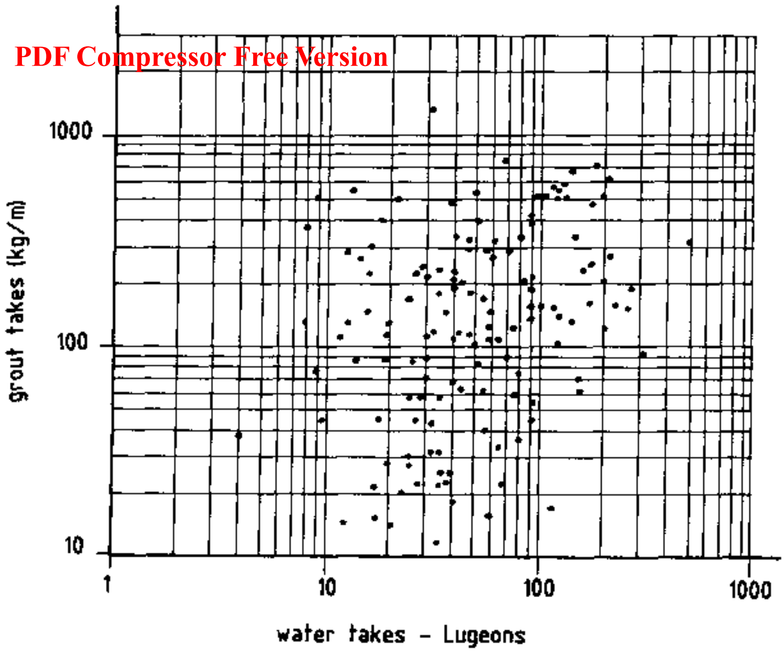


Figure 12.29. Grout take vs Lugeon value (Jawantzky in Ewart 1985).

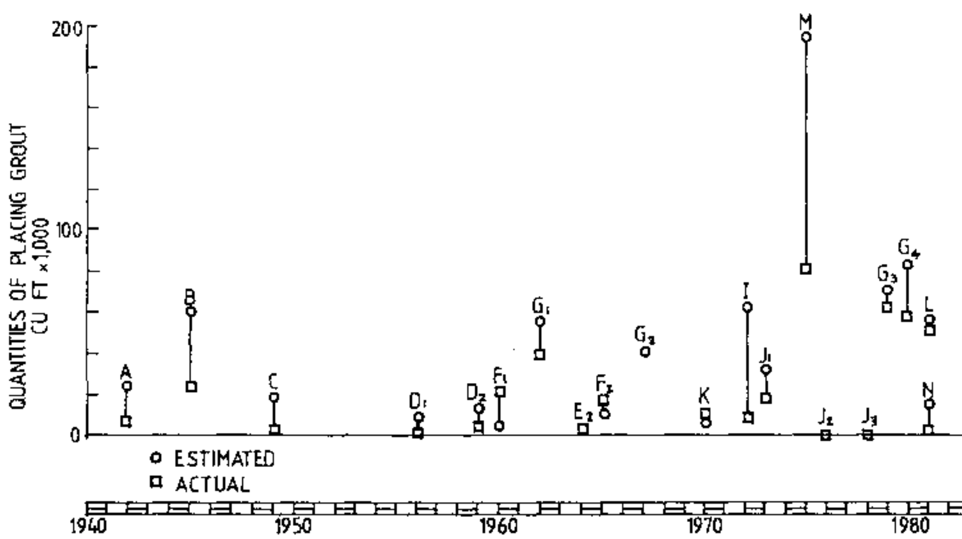


Figure 12.30. Estimated and actual grout takes in dams constructed by US Corps of Engineers (1984).

12.4 CHEMICAL GROUTS IN DAM ENGINEERING

12.4.1 *Types of chemical grouts and their properties*

There are two distinct types of chemical grouts

Group A – colloidal solutions or prepolymers, e.g. silica gel, ligno chrome gel, tannins, organic or mineral colloids, polyurethane

Group B – are solutions, e.g. acrylamide, phenoplast, aminoplast

The hydraulic behaviour of these grout types is different:

Group A – behave as Bingham fluids with an initial shear stress required to mobilize the grout (as for cement and cement/bentonite grouts)

Group B – behave as Newtonian fluids – as for water but with a higher viscosity

Figure 12.31 shows typical flow properties of grouts. Note that the viscosity measured (in centipoise) is the slope of the graph at any point; the apparent viscosity is the slope of the line passing through the point (e.g. Point A) and the zero point (rather like a secant modulus compared to a tangent modulus for true viscosity). The intercept of the curve on the shear stress axis is the 'yield stress,' or 'yield value,' (also called 'cohesion' by Lombardi 1985 by analogy with the shear strengths of soils). The value quoted will depend on the shear rate and may be determined by extrapolation of the straight line portion of the curve as shown in Figure 12.31.

Most methods of measuring will give the apparent viscosity, rather than the yield stress and true viscosity and this must be taken into account. The authors' experience is that, with cement grouts, the method proposed by Nguyen & Bogor (1983, 1985) which uses a very sensitive vane shear type apparatus is the most successful.

The viscosity is dependent on the concentration of the grout, and the time after mixing. Figures 12.32 and 12.33 show these effects.

The gel time is dependent on time and temperature, and on additives which are deliberately used to control it. This is discussed in Littlejohn (1985) and Karol (1985).

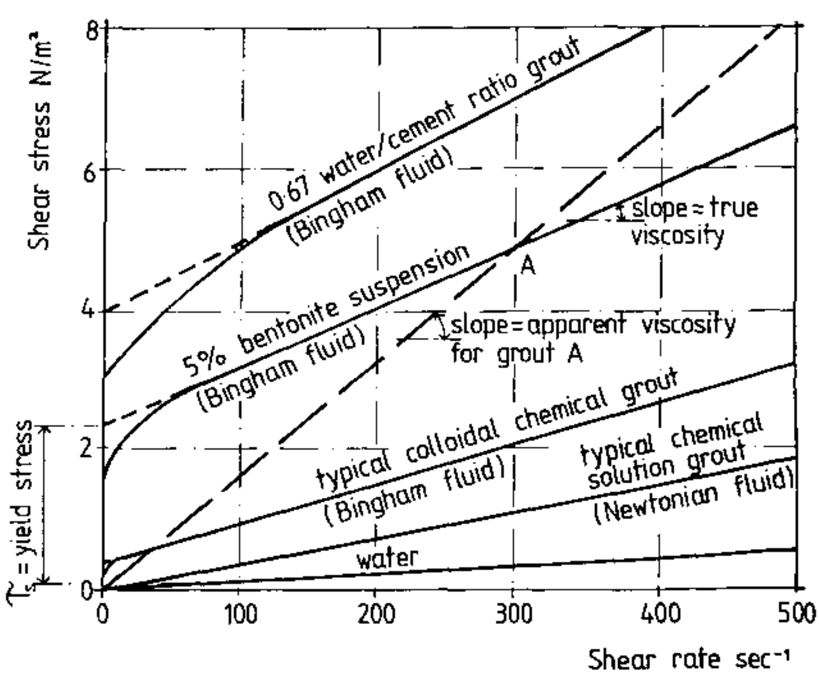


Figure 12.31. Typical flow properties for grouts (adapted from Littlejohn 1985).

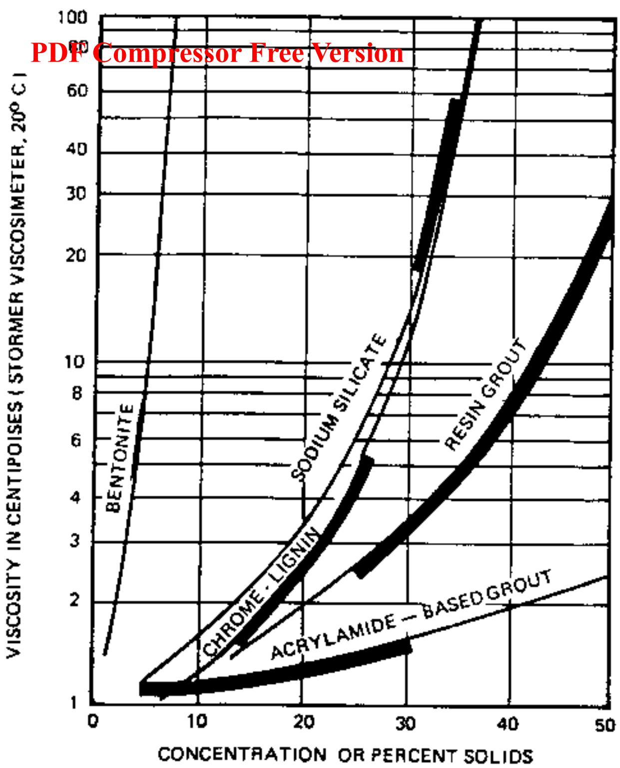


Figure 12.32. Viscosity (apparent) as a function of grout type and concentration (Karol 1983).

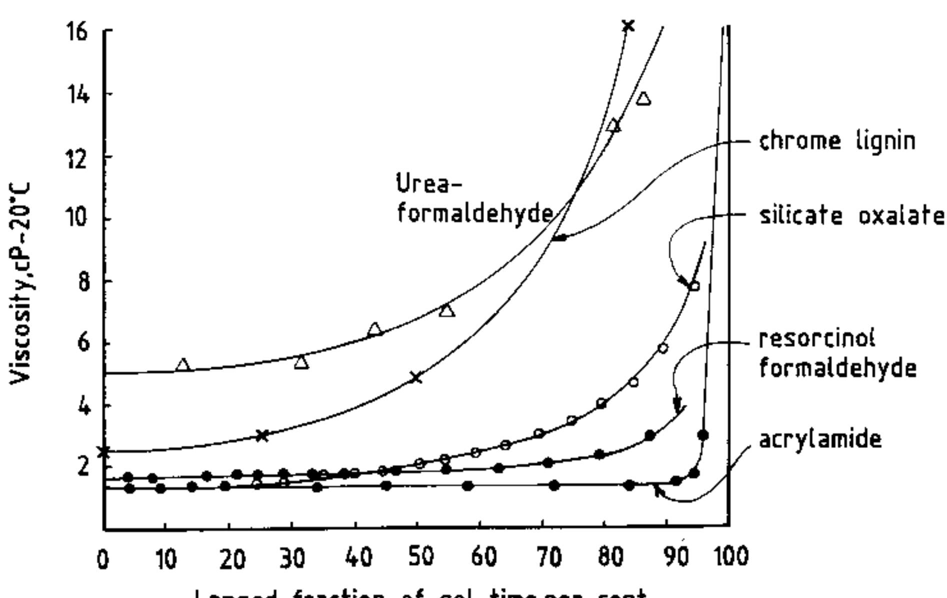


Figure 12.33. Viscosity (apparent) as a function of time (Littlejohn 1985).

Table 12.10. Some chemical grouts and their properties (Brett 1986).

Grout type	Corrosivity or toxicity	Viscosity	Strength
Silicates			
Joosten process	Low	High	High
Siroc	Medium	Medium	Medium/high
Silicate/Bicarbonate	Low	Medium	Low
Lignosulphates			
Terra firma	High	Medium	Low
Blox-all	High	Medium	Low
Phenoplasts			
Terranier	Medium	Medium	Low
Geoseal	Medium	Low/medium	Low
Rocagil	Medium	Medium	Low
Aminoplasts			
Herculon	Medium	Medium	High
Cyanaloc	Medium	Medium	High
Acrylamides			
AV-100	High	Low	Low
Rocagil BT	High	Low	Low
Nitto SS	High	Low	Low
Terragel	High	Low	Low
Poly acrylamides			
Injectite 80	Low	High	Low
Acrylate			
AC-400	Low	Low	Low
Polyurethane			
CR-250	High	High	High
CR-260	High	Medium	High
TACSS	High	High	High

There are many types of grout available and new products are coming onto the market all the time. Tables 12.10 and 12.11 list some of the more common grouts and their properties.

Karol (1982, 1983) and Littlejohn (1985) discuss chemical properties of grouts in more detail.

12.4.2 *Grout penetrability in soil and rock*

Grout penetration is dependent upon

(i) Whether penetration is by permeation (impregnation) in the voids in the soil, or by causing hydraulic fracture by exceeding the *in situ* horizontal stresses (0.5-2 times overburden pressure), or a combination of both. Australian, and general overseas (except French) practice is to limit pressures to the permeation phase.

(ii) The viscosity properties of the grout, and then the pressure and time for which grouting proceeds.

For Bingham fluids, there is a limiting radius to which grout can be pumped because of the shear stress required to mobilize the grout. Littlejohn (1985) indicates that in soil this can be estimated from

Table 12.11. Chemical grouts – A summary of properties.

Grout type	Fluid behaviour	Typical viscosity centipoise	Gel time	Stability	Examples	Comments
Sodium silicate	Newtonian initially, then Bingham	3-4 for permeability reduction 10 for strength. Viscosity increases as grout gels	30-60 minutes	1) Undergoes syneresis (loss of water and shrinkage on gelling. 2) Unstable in alkaline environment	Joosten process	Non toxic and not an environmental hazard. Syneresis less of a problem in finer soils (sand and silt). Limit is fine-medium sand. Krizak (1985) tests showed up to 10 or 100 times increase in permeability with time under high gradient seepage
Acrylamide	Newtonian	Less than 2	Gel time controlled by additives NaCl in groundwater may lessen gel time	Permanent	AM9 (now outdated because of toxicity) Injectite 80 (has higher viscosity)	AM9 was one of the most popular grout for a long time. Good permeability and permanence
Phenoplast	Newtonian	1.5 to 3 initial and constant until gel starts	Gel time controlled by additives	Permanent except under alternating wet/dry conditions	e.g. Resorcinol plus formaldehyde + NaOH e.g. Geoseal	Medium toxicity. Geoseal 2-10 centipoise used for dams at Worsley (Brett (1986)
Aminoplast	Newtonian	5, up to 10-20 with additives to stabilize gel time	Gel time controlled	Only gel in acid (pH < 7) environment. Permanent except in wet/dry conditions	Based on urea and formaldehyde	Toxic and corrosive prior to gelling (acid catalyst)
Acrylate polymer	Newtonian	2	Gel time controlled	Permanent	AC-400	Replaces AM9. Only 1% as toxic. Krizek & Perez (1985) tests showed it did not deteriorate with time under high (100) gradients

$$RL = \frac{\delta_w gHd}{4\tau s} + r$$

where RL = limiting radius of penetration
δ_w = density of water
g = acceleration due to gravity
H = hydraulic head
d = effective diameter of the average pore
τs = Bingham yield stress
r = radius of spherical injection source = $\frac{1}{2}\sqrt{LD}$ where L = length of hole, D = diameter.

and

$$d = 2\sqrt{\frac{8\mu k}{\delta_w g n}}$$

where μ = grout viscosity in centipoise
k = permeability of soil
n = porosity of soil.

In jointed rock, with a joint aperture of 2a, Lombardi (1985) indicates that the maximum radius is given by

$$R_{max} = \frac{P_{max}\, a}{C}$$

(see Section 12.2.4.5)

This is equivalent to

$$RL = \frac{Ha}{\tau s}$$

For Newtonian flow, in uniform isotropic soil from a spherical source, Littlejohn (1985) indicates that:

$$H = \frac{Q}{4nk}\left[\mu\left(\frac{1}{r} + \frac{1}{R}\right) + \frac{1}{R}\right]$$

where Q = flow rate at radius of penetration R and the time for grout to penetrate

$$t = \frac{nr^2}{kH}\left[\frac{\mu}{3}\left(\frac{R^3}{r^3} - 1\right) - \left(\frac{\mu - 1}{2}\right)\left(\frac{R^2}{r^2} - 1\right)\right]$$

i.e. time is proportional to R^3, this dictates that relatively close hole spacings are used for economic grout times, e.g. 0.5 to 2.5 m.

There are various tables and graphs indicating the types of soils and soil permeability that can economically be grouted, e.g. Littlejohn (1985) suggests Table 12.12 and Figure 12.34.

Karol (1985) suggests that based on a review of the literature, Figure 12.35 is a conservative assessment of groutability by permeation i.e. without fracturing.

Caron (1982) suggests Table 12.13 as a basis for determining whether grouting was practicale, depending on ground conditions and the type of grout.

Table 12.12. Grouting limits of common mixes (Littlejohn 1985).

Types of soils	Coarse sands and gravels	Medium to fine sands	Silty or clayey sands, silts
Soil characteristics			
Grain diameter	$D_{10} > 0.5$ mm	$0.02 < D_{10} < 0.5$ mm	$D_{10} < 0.02$ mm
Specific surface	$S < 100\ cm^{-1}$	$100\ cm^{-1} < S < 1000\ cm^{-1}$	$S > 1000\ cm^{-1}$
Permeability	$K > 10^{-3}$ m/s	$10^{-3} > K > 10^{-5}$ m/s	$K < 10^{-5}$ m/s
Type of mix	Bingham suspensions	Colloid solutions (gels)	Pure solutions (resins)
Consolidation grouting	Cement ($K > 10^{-2}$ m/s) aerated mix	Hard silica gels double shot Joosten (for $K > 10^{-4}$ m/s) Single shot: Carongel, Givanol, Siroc	Aminoplastic, phenoplastic
Impermeability grouting	Aerated mix, bentonite gel, clay gel, clay/cement	Bentonite gel, lignochromate, light carongel, soft silicagel, vulcanisable oils, others (terranier)	Acrylamids, aminoplastic, phenoplastic

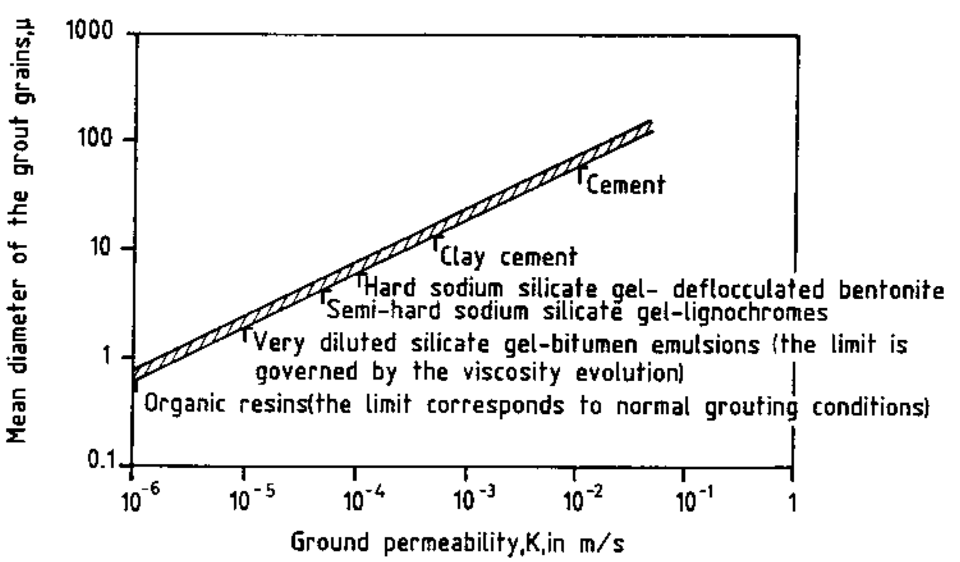


Figure 12.34. Limits of injectability of grouts based on the permeability of sands and gravels (Littlejohn 1985).

It is hard to envisage a case where grouting of soils with a permeability around 10^{-7} m/sec would be warranted or would result in significant reduction in permeability.

12.4.3 *Grouting technique*

The basic approach to use of chemical grouts is similar to that for cement grouts:
- grout is injected into holes under pressure;
- holes are drilled and grouted in stages to achieve a desired Lugeon or permeability closure.

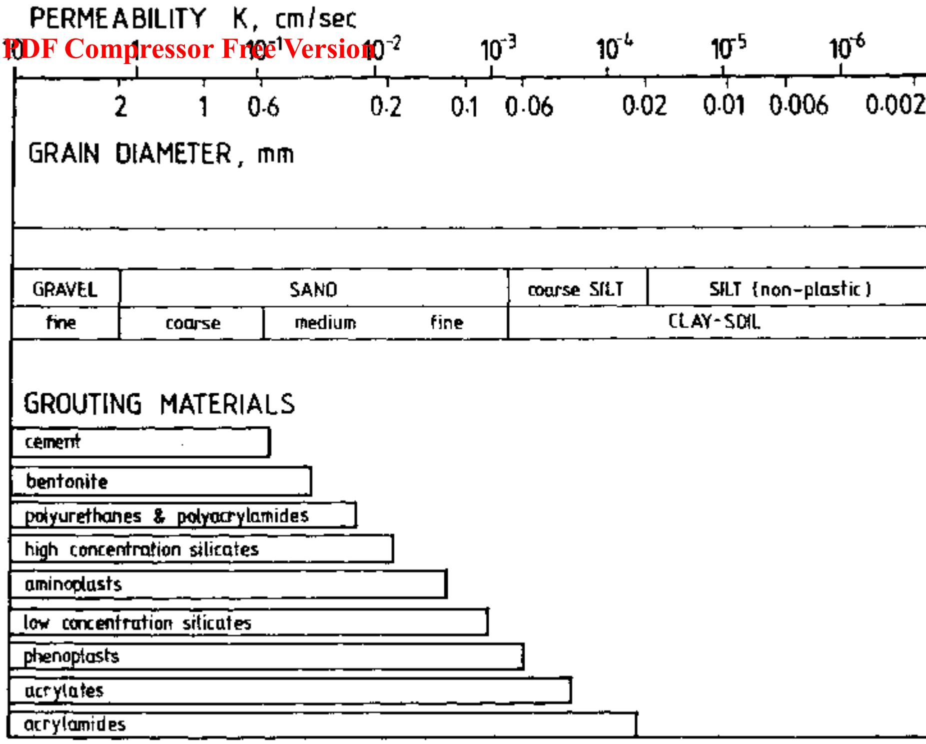


Figure 12.35. Limits of groutability (based on Karol 1985).

Table 12.13. Limits of grouting (Caron 1982).

Type of grouting method	Type of ground: Coherent fissured soils: Large medium fissures $K > 5 \times 10^{-7}$ m/sec	Very fine fissures $K < 5 \times 10^{-7}$ m/sec	Loose soils: Coarse and medium $K > 10^{3}$ m/sec	Fine $K < 10^{-3}$ m/sec
By fracturing	–	–	C	C
By impregnation	C	CG	C	CG
By fracturing	–	–	–	CG

C = cement based grout; CG = chemical grout; – = no case for grouting.

When grouting in rock, in which the holes remain open and packers can be set, downstage grouting with packer would normally be adopted. However, in most chemical grouting operations, either extremely weathered rock (i.e. virtually soil properties) or soil is being grouted, and it is necessary to support the hole from collapse and use different methods to inject the grout.

Figure 12.36 shows the tube-a-manchette ('pipe with sleeves') technique which is used in Europe (and Australia) for grouting with chemicals.

In this technique the hole is drilled and cased to its full depth, the hole filled with a ce-

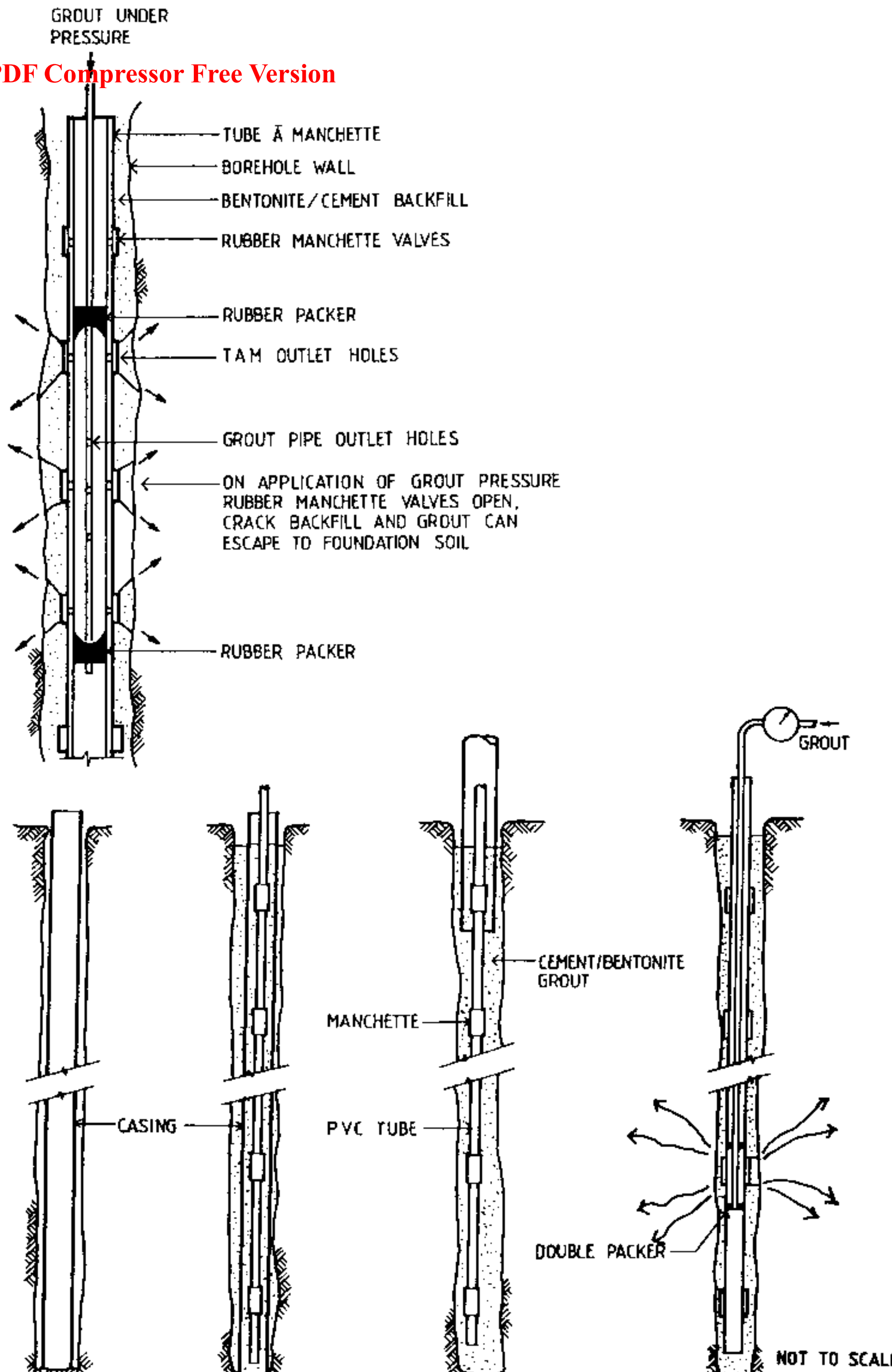


Figure 12.36. Tube-a-manchette grouting system (adapted from Brett 1986).

ment/bentonite grout and the tube a manchette installed. This consists of a 40 to 60 mm diameter PVC tube, with 6 mm diameter boreholes in the wall of the tube at 300 or 333 mm intervals. The holes are covered by a rubber sleeve.

Once installed, the casing is withdrawn and the grout allowed to 'set' to give a low strength, relatively brittle grout.

The grouting operation is carried out by lowering a double packer to isolate one set of outlet holes as shown in Figure 12.36, setting the packers, then applying the grout pressure. The pressure of the grout lifts the rubber sleeve, fractures the bentonite-cement grout and allows the chemical grout to penetrate into the soil or weathered rock.

Some idea of pre-grouting permeability can be obtained from the grout flow rate and pressure, but as explained by Caron (1982) this may not be accurate because of unquantifiable pressure losses through the fractures in the cement/bentonite grout.

Caron (1982) indicates that when grouting in sands and gravels practice differs from country to country. In USA and Japan, grouting is commonly from the bottom of the casing used to support the hole, with the casing gradually withdrawn. For a two-shot grout, such as sodium silicate/calcium chloride, the base grout (sodium silicate) is injected as the casing is lowered, and the reactant grout (calcium chloride) as the casing is withdrawn (as shown in Fig. 12.37). This method does allow checking of permeability before grouting (but only crudely out the bottom of the casing) and gives poorer control than the tube-a-manchette system.

As shown in Figure 12.38, grouting should be carried out on a 'closure' basis in depth and plan and at least 3 lines of holes should be used to achieve this. Initial spacing should be 2 to 3 times the anticipated final spacing.

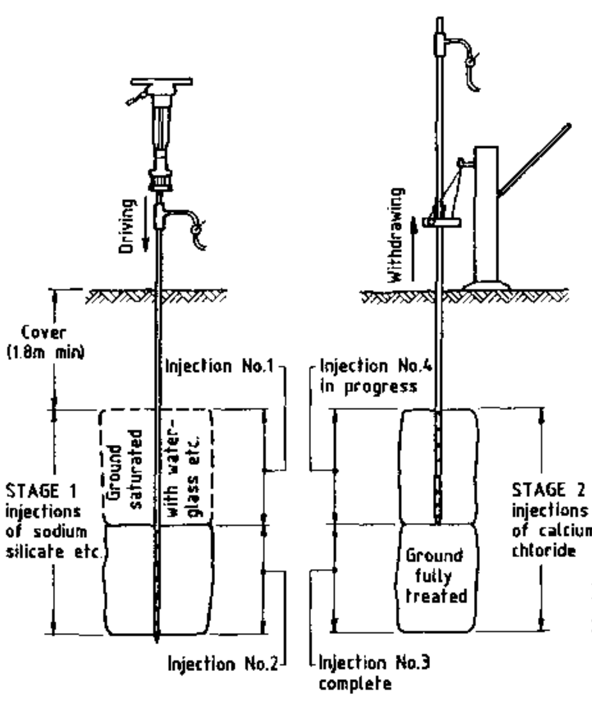


Figure 12.37. Open bottom hole method of grouting (Littlejohn 1985).

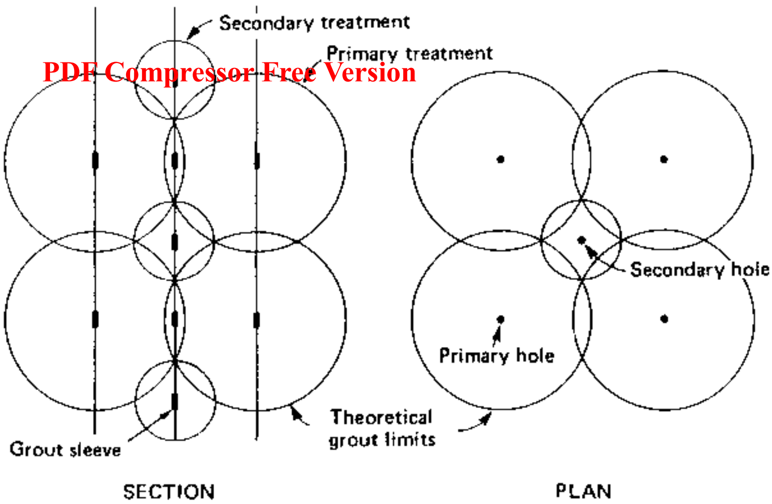


Figure 12.38. Staging of chemical grouting (Littlejohn 1985).

Table 12.14. Relative material costs of grout formulations (Littlejohn 1985).

Formulation	Relative cost of materials
Cement-bentonite	
w/c = 3.5% bentonite by wt. of water	1.0
w/c = 2.3% bentonite by wt. of water	1.3
w/c = 1.1% bentonite by wt. of water	2.3
Cement	
(w/c = 0.5)	3.4
Silicate-bentonite	
20% bentonite, 7% silicate (by wt. of water)	1.3
Silicate-chloride (Joosten)	4.0
Silicate-ester	
37% silicate, 4.4% ester (by volume)	5.0
47% silicate, 5.6% ester (by volume)	6.5
Silicate-aluminate	
46% silicate, 1.4% aluminate (by weight)	5.0
Phenol-formaldehyde	
13% (by volume)	10.5
19% (by volume)	15.3
Acrylate	
10% (by weight)	18.5
Resorcinol-formaldehyde	
21% (by volume)	23.0
28% (by volume)	31.0
Polyacrylamide	
5% (by volume)	20.0
10% (by volume)	40.0

12.4.4 *Applications to dam engineering*

Chemical grouts are expensive compared to cement or cement bentonite grouts. Table 12.14, reproduced from Littlejohn (1985), gives relative costs of several different grouts. These are material costs, and do not include drilling or injecting the grout.

As a result, the use of chemical grouts will be restricted to those cases where seepage is critical and not controllable by cement grouts, or in cases of remedial works, particularly in alluvial (soil) foundations where cement grouts are not applicable.

As in the use of cement grouts, there is a limit to the permeability which can be achieved by chemical grouting. Littlejohn (1985) suggests that practical minimum average permeabilities are:

– 1×10^{-7} m/sec in coarse sands and gravels (and only after 'sophisticated chemicals, careful injection procedures and close supervision');

– 1×10^{-8} m/sec in 'fissured' rock.

Since the grout curtain width is likely to be narrow (say 1 to 1.5 m maximum per row of holes), significant seepage reduction will only result when the original permeability is relatively high compared to the grout curtain (see Figure 12.19).

Brett (1986) presents a plausible argument for two or three stage cement/bentonite, followed by chemical grouting. The former is to fill the larger voids and hence reduce the cost of chemicals. Figure 12.39 illustrates this approach.

In many cases where in the past chemical grouts have been used to reduce leakage under dams on alluvial sand and gravel foundations, it would now be more reliable and less costly to use slurry trench or diaphragm wall cutoffs constructed of bentonite or bentonite-cement. These give a more controllable uniform width of low permeability cutoff than can be achieved by grouting and would generally be the authors' preference (see Chapter 9 for more discussion on this matter).

Examples of the use of chemical grouting in dams are given in Littlejohn (1985), Brett & Osborne (1984), Davidson & Perez (1982), Bell (1982) and Graf et al. (1985).

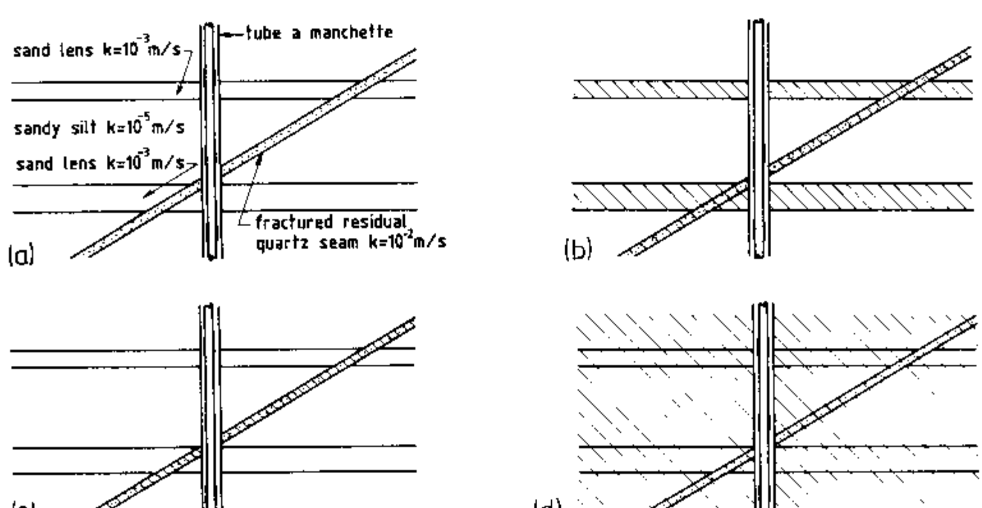


Figure 12.39. Multiple grout system (Brett 1986) a) ground properties; b) first stage cement grout; c) second stage bentonite grout; d) third stage chemical grout.

CHAPTER 13

Embankment details

13.1 FREEBOARD

13.1.1 *Definition and overall requirements*

a) Freeboard. The freeboard for a dam is the vertical distance between a specified reservoir water surface level and the top of the dam, without allowance for camber of the top of the dam.

b) Normal freeboard is the vertical distance between the top of the dam without allowance for camber, and the normal reservoir full supply level.

c) Minimum freeboard is the vertical distance between the top of the dam without allowance for camber, and the maximum reservoir water surface that results from routing the inflow design flood through the reservoir.

The objective of having freeboard is to provide assurance against overtopping resulting from (USBR 1981)

– wind set up,
– wave runup,
– landslide and seismic effects,
– settlement,
– malfunction of structures,
– other uncertainties in design, construction and operation.

Other factors which may influence the selection of freeboard include (ANCOLD 1986)

– reliability of design flood estimates,
– assumptions made in flood routing,
– type of dam and susceptibility to erosion by overflow,
– potential changes in design flood estimates, either through changes in flood estimation techniques or due to changed catchment conditions.

ANCOLD (1986) recommend that three cases should be considered to assess freeboard for flood surcharge and wave runup:

1) Maximum reservoir water level (i.e. stillwater level resulting from flood routing of the design flood and including any condition of malfunction or contingency judged to be a normal risk), in combination with wind effects due to a moderate wind.

2) Intermediate reservoir water level (i.e. stillwater level resulting from routing of a flood of reasonable probability, including any condition of malfunction or contingency judged to be an abnormal risk), in combination with wind effects due to a high velocity wind (i.e. low probability).

3) Normal reservoir water level (no flood inflow), i.e. usual full supply level, in combination with:

– wind effects due to extreme wind;

– seismically-generated wave action, or landslide-generated displacement of reservoir volume and associated wave action.

ANCOLD (1986) indicate that it is the designer's responsibility to define the generalised expressions such as normal, abnormal, moderate, reasonable, etc. in terms of probabilities. The objective is to select combinations of factors to give comparable composite probabilities of exceedence and avoid superimposing risks of very low probability. In assessing probabilities of coincidence of high reservoir level and high wind, meteorological advice should be sought.

The freeboard adopted in a particular case will be the largest of the values given for the cases selected and will take account of the different requirements for concrete and embankment dams. However, landslide or seismically generated effects should be considered, if, in the designer's judgement based on geological advice, such events have been identified as real possibilities.

These requirements are broadly consistent with those of the USBR (1981) who indicate that normal and minimum freeboard requirements should be evaluated, and the freeboard resulting in the higher top of dam elevation adopted. The requirements are:

'Minimum freeboard combinations – the following components when they can reasonably occur simultaneously, should be combined to determine the total minimum freeboard requirement:

a) wind-generated wave runup and setup for a moderate wind,

b) malfunction of spillway and/or outlet for a moderate wind,

c) settlement of embankment and foundation not included in crest camber,

d) hydrologic uncertainties resulting from inadequate data base,

e) landslide-generated water waves and/or displacement of reservoir volume (only cases where landslides are triggered by the occurrence of higher water elevations and intense precipitation associated with the occurrence of the inflow design flood.

Normal freeboard combinations – the most critical of the following two combinations of components should be used for determining normal freeboard requirements:

a) (1) Wind-generated wave runup and setup for maximum wind; (2) settlement of embankment and foundation not included in crest camber.

b) (1) Landslide-generated water waves and/or displacement of reservoir volume; (2) settlement of embankment and foundation not included in crest camber; (3) settlement of embankment and foundation from maximum credible earthquake.

Intermediate freeboard combinations – a reasonable combination of components should be determined on a case by case basis by the designer. This would apply to cases where there are exclusive flood control storage allocations.'

The USBR (1981) indicate that for ungated spillways, the minimum freeboard for embankment dams should be 0.9 m. For gated spillways they list additional factors which must be considered. They indicate that this may be relaxed somewhat for existing dams where operational factors and settlement will be better known than for a new dam.

13.1.2 *Estimation of wave runup freeboard for feasibility and preliminary design*

For feasibility and preliminary design studies the USBR (1981) indicate that the method outlined in USBR (1977) can be adopted.

Table 13.1. Freeboard requirements for preliminary studies (USBR 1977).

Largest fetch (km)	Normal freeboard (m)	Minimum freeboard (m)
Less than 1.6	1.2	0.9
1.6	1.5	1.2
4	1.8	1.5
8	2.4	1.8
16	3.0	2.1

This is based on a wind velocity of 160 km/hr (100 miles per hour) for determination of normal freeboard, and 80 km/hr (50 miles per hour) for minimum freeboard. The effect of wind setup is ignored. For rip-rapped slopes the freeboard requirements are as tabulated in Table 13.1.

For dams with a smooth pavement or soil cement upstream slope, depending on the smoothness of the surface, freeboard of up to 1.5 times those shown in Table 13.1 should be used.

13.1.3 *Estimation of wind setup and wave runup for detailed design*

For detailed design the method outlined in USBR (1981) can be used. This is based on US Corps of Engineers (1976) and Saville et al. (1962). The following summarizes the main steps in the method.

13.1.3.1 *Fetch*

In reservoirs, fetches are limited by the land surrounding the body of water. The shorelines are irregular and an effective fetch is calculated from

$$F = \frac{\Sigma x_i \cos \alpha_i}{\Sigma \cos \alpha_i}$$

where α_i = angle between the central radial from the dam and radial i

x_i = length of projection of radial i on the central radial.

A trial and error approach should be used to select the critical position on the dam and direction of the central radial to give the maximum effective fetch.

The radials spanning 45° on each side of the central radial should be used to compute the effective fetch. Figure 13.1 shows an example.

13.1.3.2 *Design wind*

Design wind estimates should be obtained from the bureau of meteorology or equivalent organisation. The critical wind duration depends on the fetch – longer durations are critical for longer fetches. The estimates should allow for local topographic effects.

USBR (1981) indicate that for maximum freeboard, moderate winds should be used, i.e. winds that can reasonably be expected to occur concurrent with maximum reservoir flood level. If the rate of flood rise is rapid, the high wind velocity associated with the design storm may be applicable, but for a slower rate of flood rise, a lower wind velocity would be appropriate. In many cases, a wind velocity of the order of 1 in 10 year event is appropriate.

For the normal freeboard calculation, maximum expected wind velocities, duration and direction should be used. The values should exceed 1 in 100 year winds. The estimates should

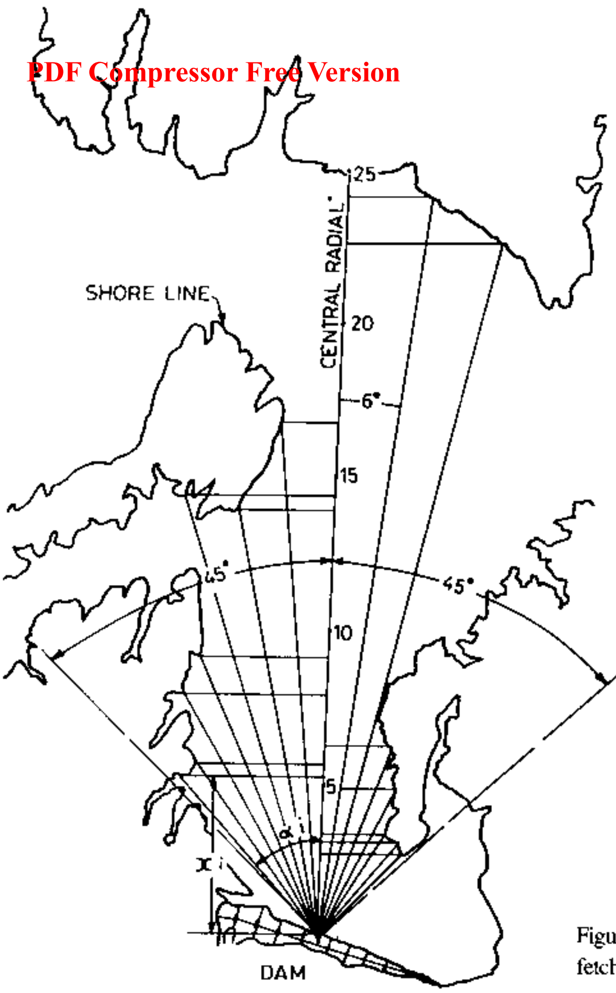


Figure 13.1. Method for calculating effective fetch (adapted from Saville et al. 1962).

Table 13.2. Wind relationships – Water to land (USBR 1981).

Effective fetch (Fe) (km)	0.8	1.6	3.2	4.8	6.4	8 (or more)
Wind velocity ratio $\left(\frac{\text{over water}}{\text{over land}}\right)$	1.08	1.13	1.21	1.26	1.28	1.30

be adjusted over land to over water wind velocity by applying the correction factors listed in Table 13.2.

The design wind velocity and duration are determined iteratively, by using bureau of meteorology estimates of wind speed vs duration, and applying these to wind velocity vs fetch vs minimum duration shown in Figure 13.2.

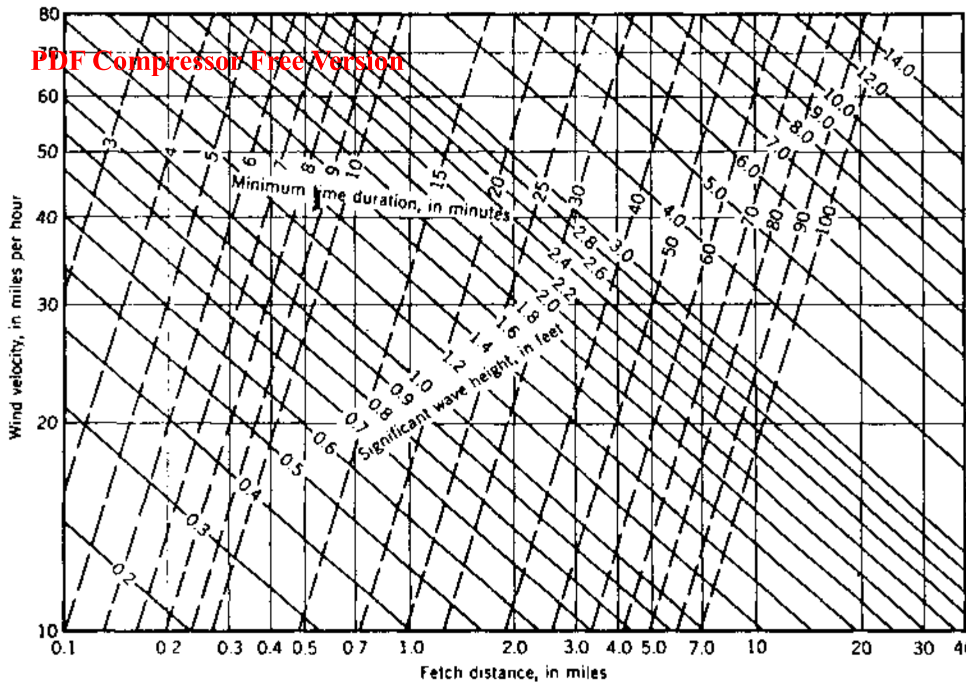


Figure 13.2. Wave heights and minimum duration wind (Saville et al. 1962 and USBR 1981). Note 1 mile per hour = 1.6 km per hour, 1 foot = 0.3 m.

13.1.3.3 *Wave height*

For the estimation of the minimum freeboard the significant wave height in metres (Hs), which is the average of the highest one-third of the waves in the wave spectrum should be used. Hs can be estimated from Figure 13.2. The wave period (T) can be estimated from Figure 13.3.

The deep water wave length in feet can then be computed from

$$L = 1.56 T^2$$

in which T = wave period in seconds.

This equation is valid for most reservoirs where the reservoir is relatively deep compared to the wind generated wave length (i.e. water depth > 0.5L). For the normal freeboard computation, the runup should be calculated using the average of the highest 10% of waves, which is 1.27 Hs.

13.1.3.4 *Wave runup*

Runup (R_s) from a significant wave on an embankment with rip-rap surface underlain by an earthfill embankment can be calculated from

$$R_s = \frac{H_s}{0.4 + (H_s / L)^{0.5} \cot \Theta}$$

where R_s = runup height in metres

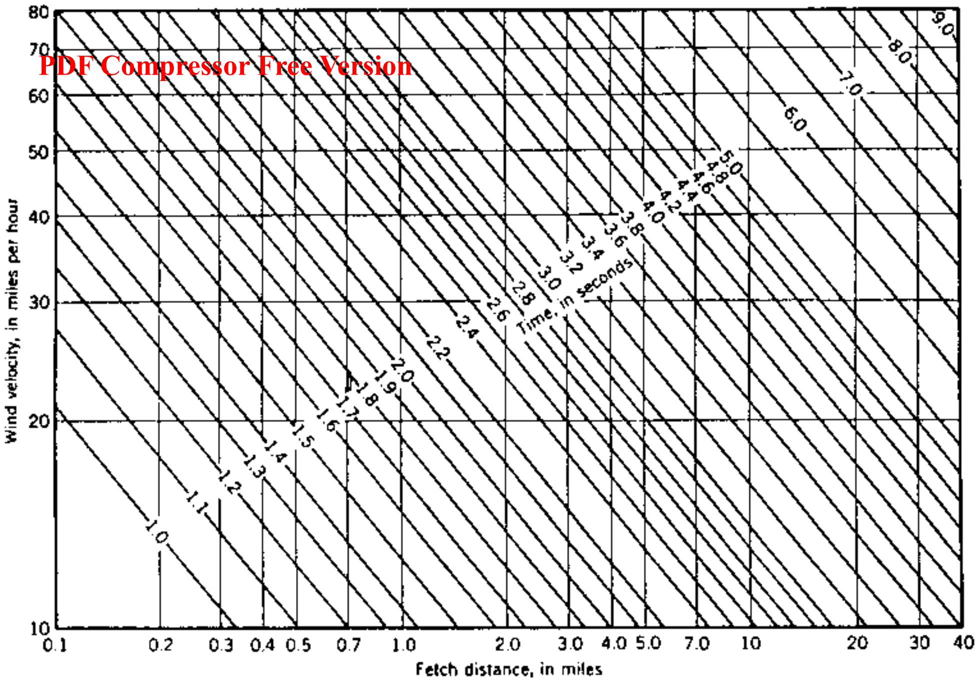


Figure 13.3. Wave periods (Saville et al. 1962 and USBR 1981).

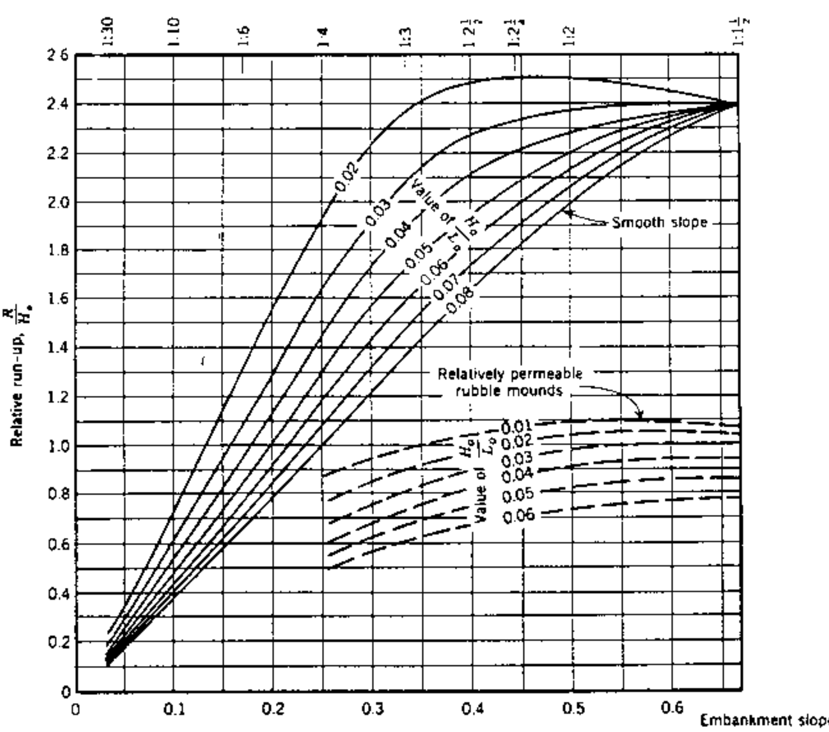


Figure 13.4. Wave runup ratios versus wave steepness and embankment slopes (Saville et al. 1962). Note: R = runup height; Ho = wave height; Lo = wave length.

H_s = significant wave height in metres
L = wave length in metres

Θ = angle of upstream face of the dam to the horizontal.

Rockfill dams act like permeable rubble mounds and have a different effect on energy dissipation to rip-rap placed on an 'impervious' earthfill embankment. For these dams the 'relatively permeable rubble mound' curves in Figure 13.4 should be used.

For smooth impermeable slopes of concrete or soil cement with a water depth at the dam greater than three times the wave height, the 'smooth slope' curves in Figure 13.4 should be used. USBR (1981) indicate that these curves may underestimate runup by in the order of 15 to 20%.

For waves which are not normal to the dam, a correction should be applied to the computed runup. This is done by multiplying the runup by the cosine of the angle between the wave propagation direction and a line normal to the dam, provided the angle is less than about 50°.

13.1.3.5 *Wind setup*

The wind setup (S) in metres is

$$S = \frac{U^2F}{62000\,D}$$

in which U = design wind velocity over water in km per hour
F = wind fetch in km, normally equals 2Fe
D = average water depth along the central radial in metres.

The wind setup should be added to the wave runup. It will be noted that apart from long shallow reservoirs subject to high winds the effect is very small.

13.2 EMBANKMENT CREST DETAILS

13.2.1 *Camber*

Embankment dams are subject to settlement after construction. In the case of earthfill this can be related to consolidation of the earthfill as pore pressures reach equilibrium but post construction settlement also occurs in rockfill, as high contact pressures between particles of rock are crushed with time.

Figure 13.5 shows measured settlements for compacted and dumped rockfill dams.

It will be seen that the settlements for compacted rockfill are small – between 0.05 and 0.25% of the height of the dam. Sherard & Cooke (1987) suggest that settlement will generally be 0.15 to 0.25% in 100 years. Dumped rockfill settles considerably more with a long term creep component.

Sherard et al. (1963) indicate that earthfill dams will generally settle 0.2 to 0.4% of their height.

To maintain freeboard it is common to build the crest of the dam higher than the design level, i.e. to provide camber. The camber relating to settlements of the dam foundation is best calculated using conventional soil and rock mechanics principles.

The camber relating to settlement of the embankment is best estimated by comparison with performance of other dams. Hence a fairly conservative approach would be to assume settlement of 0.25% for compacted rockfill, and 0.5% for earthfill dams.

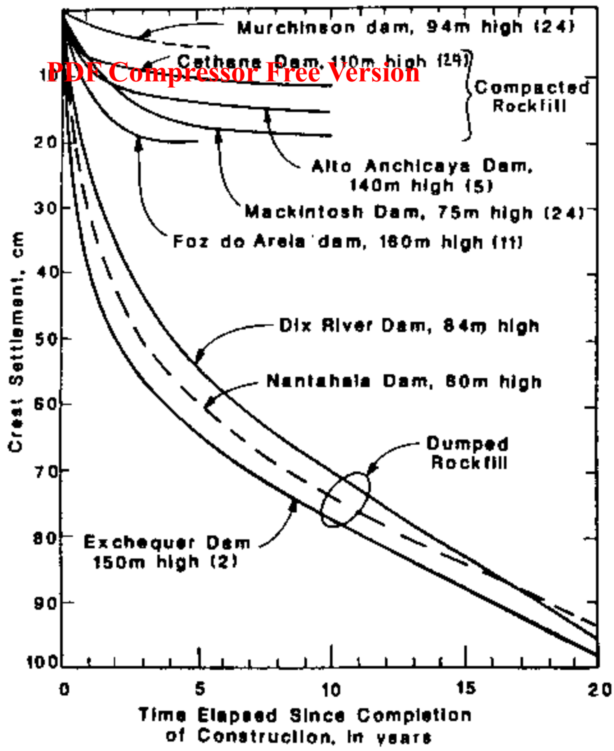


Figure 13.5. Comparison of crest settlements for compacted and dumped rockfill dams (Sherard & Cooke 1987).

As pointed out by Sherard et al. (1963) it is preferable to err on the conservative side because in addition to the danger in association with loss of freeboard, even the slightest sag in the crest can be recognised.

USBR (1977) suggest using 1% camber, but in most cases that would seem excessive.

The camber is usually provided by oversteepening the upper slopes of the embankment. This may involve steepening by up to 0.25H:1V.

The camber is varied in a smooth curve or series of straight lines along the dam crest, with camber provided proportional to the height of the dam above the foundation.

13.2.2 *Crest width*

The crest width has no appreciable influence on the overall stability of a dam and is determined by the minimum practicable width for construction purposes, and possible roadway requirements.

USBR (1977) give an empirical formula relating crest width to dam height, i.e.

$$W = \frac{Z}{5} + 3.3\,\text{metres}$$

where W = width of crest in metres

Z = height of dam above stream bed in metres.

This may be used as an overall guide for earth and earth and rockfill dams, but there is no real reason why the crest width should be a function of height.

Figures 1.4 to 1.13 in Chapter 1, and Figure 13.9, show crest details for several dams. These show how filter zones are reduced in width near the crest so that the overall width can be kept to a minimum. Any earthfill zone should be at least 3 m wide at the crest to allow compaction with normal rollers. No dam should have a crest width less than 3 m to allow vehicular access for maintenance purposes.

The crest is generally sloped towards the reservoir to stop water ponding on it, and covered with a pavement to allow vehicles to traffic the crest. The authors' experience is that where no pavement has been provided, traffic causes wheel rutting and ponding of water on the surface which is undesirable. If no pavement is provided it is wise to keep traffic off the crest by providing posts or a fence at each end.

13.2.3 *Curvature of crest in plan*

Some engineers have favoured curving the crest of earth, and earth and rockfill dams, in the upstream direction. As discussed by Wilson (1973), when the water load is imposed on a dam, the crest moves downstream as the water load is applied, and if the dam crest is curved upstream in plan, this can result in compressive strains along the axis of the dam.

These compressive strains can counter tensile strains which are induced by settlement of the dam, and hence reduce the likelihood of hydraulic fracture and leakage through the earthfill zones of the dam.

USBR (1977) suggest that for small dams the extra difficulty involved in constructing a curved axis is not warranted. Sherard (1973) indicates that the additional cost is very small, but the benefits of curvature are doubtful.

Sherard & Dunnigan (1985) indicate that with the greater confidence in the ability of well designed filters to control erosion, curvature of the dam axis is not considered necessary, even for high dams in steep valleys. The authors agree with this point of view.

13.3 EMBANKMENT DIMENSIONING AND TOLERANCES

13.3.1 *Dimensioning*

Embankment dimensioning is often done poorly by inexperienced engineers, because they fail to consider properly how the dam will be set out and constructed. Figure 13.6 shows some of the common errors. They are:

– Dimensioning height of dam rather than the reduced level of the crest including camber (RL b + camber). The height of dam varies across the valley and in any case is not known at any section prior to construction because the general foundation level is not known.

– Giving a reduced level at the base of the dam, RL t and/or depth of cutoff trench Z. These vary across the valley and are also not known prior to construction at any section. General foundation and cutoff foundation should be defined in geotechnical terms, not as levels and depths below ground surface or general excavation.

– Setting out the cutoff trench from the contact of earthfill zone with general excavation (i.e. point A) rather than as a fixed offset from the centreline. The position of A is not known prior to construction and is a variable distance from the centreline giving a 'meandering' location of the cutoff trench. The cutoff trench width is usually defined as shown, i.e. with width w at the cutoff foundation. In practice it is better to define the width D E which can be set and after general

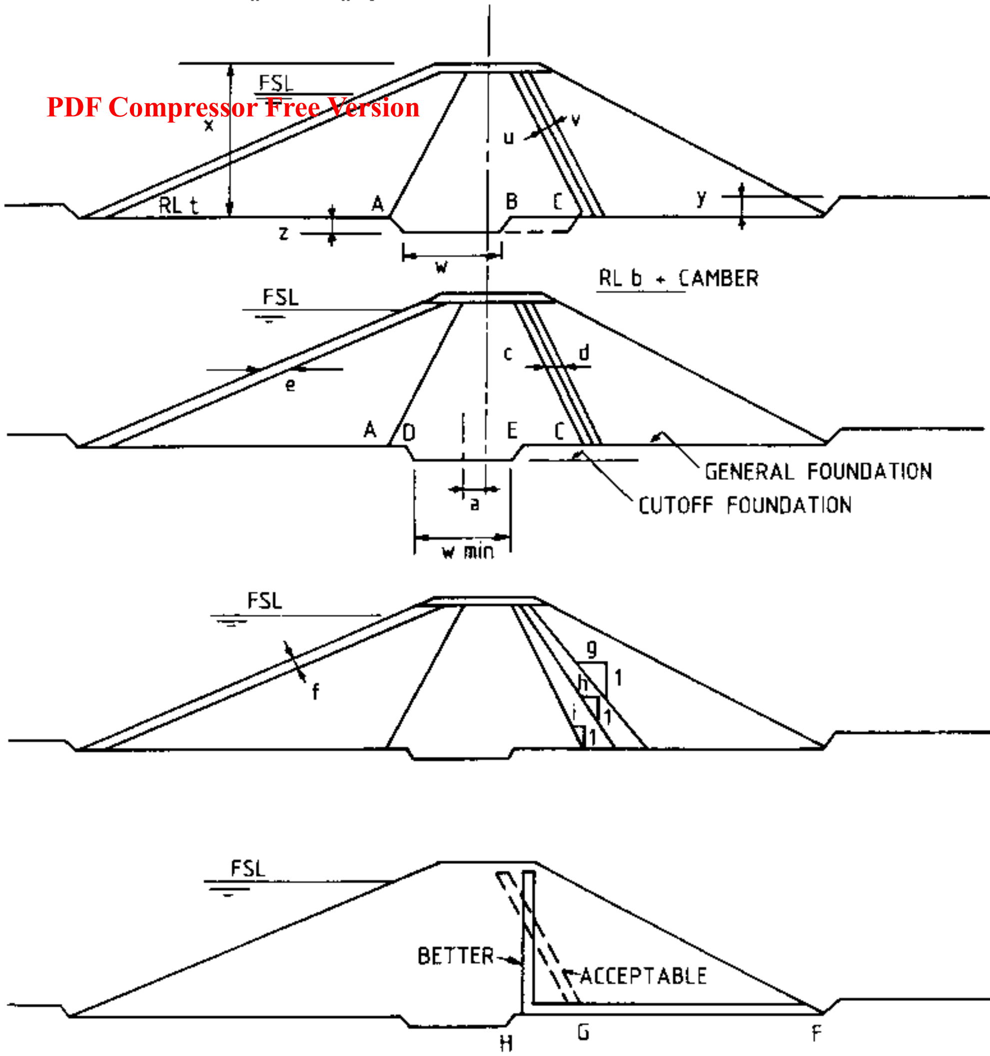


Figure 13.6. Common errors in dimensioning dams.

excavation is completed, with an overriding dimension w_{min} to cover the situation where the depth of excavation to cutoff foundation is deeper than anticipated

– Dimensioning filter and rip-rap layers as thicknesses normal to the slope (u, v and f) rather than as widths c, d and e. In most cases construction is in horizontal lifts so the widths are more appropriate

– Dimensioning filter zones with varying widths using slopes, g, h, i rather than as constant widths. There is no technical need for the filter width to increase with height, and setting out is more complicated and volume of filters are increased by using the varying widths.

– Chimney drains should be vertical rather than inclined, as they are better constructed by the digging back through compacted earthfill (see Fig. 7.16) rather than placed concurrent with the fill. It is easier to dig a vertical trench than an inclined one.

Table 13.3. Tolerances for embankment construction.

Line	Tolerances Towards axis of dam	Away from axis of dam
(a) Outside faces of dam embankment		
Crest	Zero	250 mm
Downstream slope	Zero	500 mm
Upstream slope	Zero	500 mm
(b) Division lines between zones upstream of dam axis		
Zone 1 and Zone 2A	Zero	1000 mm
Zones 3A and 3B	1500 mm	1500 mm
(c) Division lines between zones downstream of dam axis		
Zone 1 and Zone 2A	Zero	1000 mm
Zones 3A and 3B	1500 mm	1500 mm
(d) Width of filter zones and rip-rap		
Zones 2A, 2B, 2C	Plus	250 mm or 500 mm
	Minus	Zero
Zone 4	Plus	500 mm or 1000 mm
	Minus	Zero
(e) Thickness of horizontal filter drains		
Zones 2A, 2B	Plus	250 mm
	Minus	Zero

13.3.2 *Tolerances*

Specifications for construction of embankment dams usually include tolerances on the dimensions of the embankment. These reflect the need to ensure that, for example, the thickness and the widths of filter zones are maintained even if the position of the filter in the embankment varies slightly.

For the dam cross sections shown in Figure 1.3 the tolerances shown in Table 13.3 would be reasonable.

13.4 SLOPE PROTECTION

13.4.1 *Upstream slope protection*

13.4.1.1 *General requirements*

The upstream slope of earthfill or earth and rockfill dams need protection from erosion by wave action on the reservoir. Earlier dams were often protected by hand placed rock, but modern dams are generally protected by dumped rockfill, known as rip-rap.

Rip-rap comprises quarried blocks of rock which have to be:

– large enough to dissipate the energy of the waves without being displaced,

– strong enough to do this without abrading or without breaking down to smaller sizes,

– durable enough to withstand the effects of long term exposure to the weather and varying periods of inundation without becoming weaker and, hence, wearing or breaking down to smaller sizes.

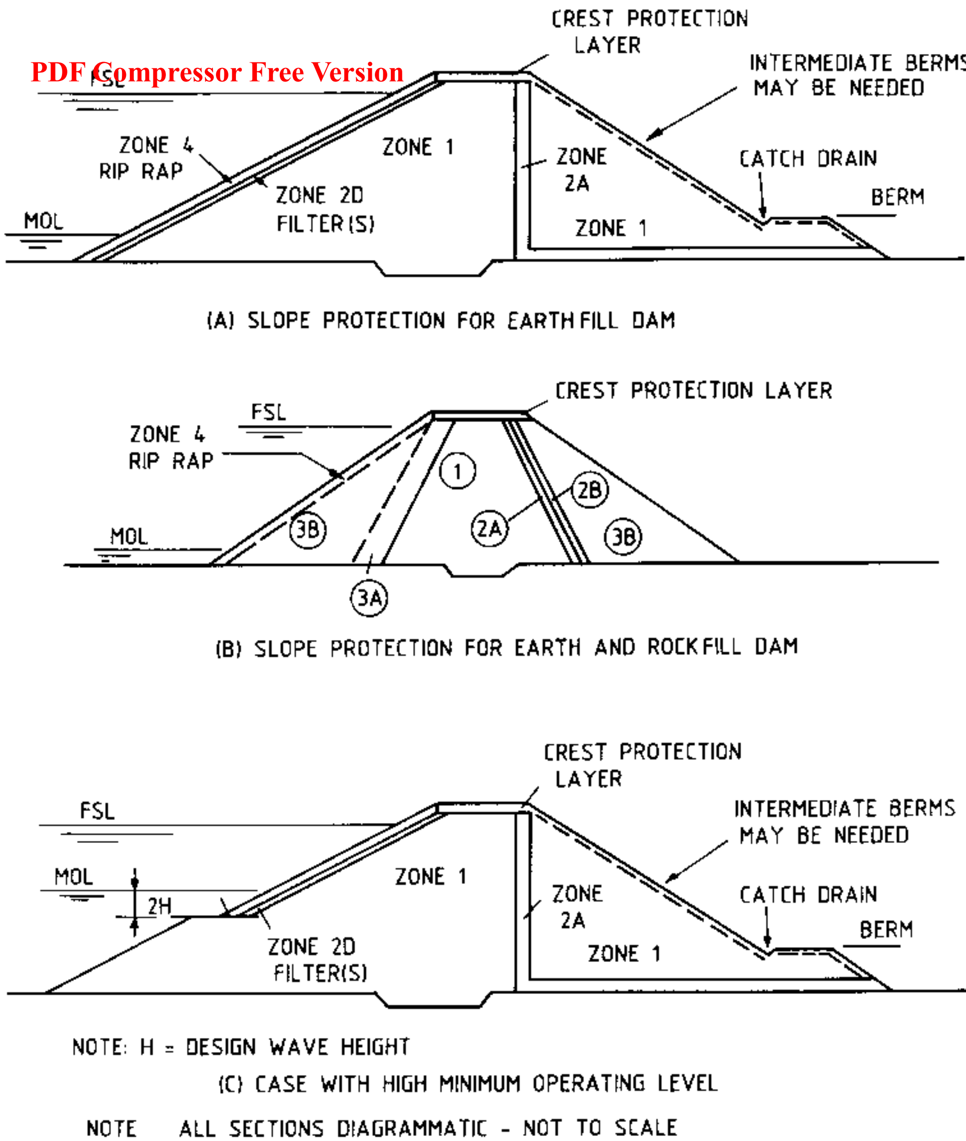


Figure 13.7. Upstream slope protection.

For earthfill dams, the rip-rap is constructed as a separate layer and may be underlain by a filter to prevent erosion of the earthfill through the rip-rap as shown in Figure 13.7A. Alternatively a wider zone of quarry run rockfill with the larger rocks pushed to the upstream slope may be adopted. For earth and rockfill dams, the rip-rap is usually obtained by pushing the larger rock from Zone 3B to the edge. Only in dams where severe wave action is anticipated and Zone 3B rockfill is too small, or not sufficiently durable, would a separate layer be placed.

Where the reservoir is operated so that the water level is maintained at a high level, ie. there is a minimum operating level well above the base of the dam, it may be possible to provide lesser or no rip-rap protection on the lower part of the dam. Because wave action affects about two times the wave height below the water level, rip-rap should be provided to minimum operating level less two times the design wave height. A small berm should be provided at this level to support the rip-rap layer and prevent undermining of the rip-rap by erosion when the reservoir first fills as shown in Figure 13.7C).

On some smaller dams, particularly those on mine sites where there is earth and rock moving equipment and waste rock readily available, rip-rap may be designed in the knowledge that damage will occur due to larger waves, or to breakdown of non durable rock, but that the damage can be readily repaired.

13.4.1.2 *Sizing and layer thickness*

The sizing of rock needed for rip-rap, the layer thickness required, and filter layer requirements are determined from the size of waves expected on the reservoir and the nature of the earthfill or rockfill under the rip-rap. The procedure recommended for sizing of rip-rap is that given in the US Corps of Engineers Shore Protection Manual (US Corps of Engineers 1984a). The steps involved are:

– Determine the design wave height H. This is taken on the average of the top 10% of the waves, which is 1.27 times the significant wave height determined as outlined in Section 13.1.3.

– Calculate the weight of the graded rock in the rip-rap from

$$W_{50} = \frac{\gamma_r H^3}{K_{RR}\,(S_r - 1)^3 \cot \Theta}$$

where W_{50} = weight in kilonewtons of the 50 percent size in the rip-rap

γ_r = unit weight of the rip-rap rock substance in kN/m^3

H = design wave height in metres

S_r = specific gravity of the rip-rap rock water relative to the water in the dam ($S_r = \gamma_r/\gamma_w$)

Θ = angle of upstream slope of the dam measured from the horizontal in degrees

K_{RR} = stability coefficient = 2.5 for angular quarried rock and non breaking waves.

The maximum weight of graded rip-rap (W_{100}) is 4 W_{50} and the minimum 0.125 W_{50}.

This is equivalent to the maximum size being 1.5 times the D_{50} size, and minimum size 0.5 times D_{50}. In practice if smaller minimum size rock is included it may be washed out under wave action. These values allow for less than 5% damage under the design wave. US Corp Engineers (1984a) give factors which allow for greater damage.

For single size rip-rap, K_{RR} is replaced by K_D with K_D = 2.4 for smooth rounded rock and 4.0 for rough angular rock. These factors assume a rip-rap layer thickness allowing two layers of rock.

The equivalent sieve size of the rip-rap is approximately 1.15 $(W/\gamma_r)^{0.33}$. Where a thin rip-rap layer is being adopted, the average layer thickness can be calculated from

$$r = n\,K_\Delta\,(W/\gamma_r)^{0.33}$$

where r = average layer thickness in metres

n = number of sub layers of rip-rap size W in the layer

K_Δ = 1.02 for smooth rounded rock = 1.0 for angular quarried rock.

For rock with a specific gravity of 2.6 the estimated sizes of rockfill are as shown in Table 13.4. The wave height shown is the 10 percentile height. Values are given for upstream slopes of 3H:1V and 2H:1V.

Table 13.4. Rip-rap size vs wave height.

Wave height (m)	Rip-rap size – Metres			
	3H:1V slope		2H: 1V slope	
	D_{50}	D_{100}	D_{50}	D_{100}
0.5	0.19	0.27	0.21	0.30
1.0	0.37	0.55	0.42	0.63
1.5	0.55	0.82	0.63	0.95
2.0	0.73	1.10	0.84	1.26
2.5	0.92	1.38	1.05	1.58

Table 13.5. Damage to rip-rap in percent as a function of design wave height (US Corps of Engineers 1984a).

H/H_{DO}	Damage (%)
1.00	0- 5
1.08	5-10
1.19	10-15
1.27	15-20
1.37	20-30
1.47	30-40
1.56	40-50

H = actual wave height; H_{DO} = design wave height.

13.4.1.3 *Selection of design wind speed and acceptable damage*

On any project, the decision has to be made as to what is an acceptable degree of damage. This will influence selection of the design wind velocity and recurrence period. A useful guide to this can be obtained by considering alternative design wind velocity recurrence intervals, e.g. 1 in 10 year, 1 in 20 year, 1 in 50 year, 1 in 100 year, and assessing the effects of using a high recurrence design velocity using the damage estimates from Table 13.5. The figures in Table 13.5 are for quarried rock rip-rap.

What damage is regarded as acceptable will depend on the importance of the structure, and the ease of access for repairs.

13.4.1.4 *Rock quality and quarrying*

The rock used for rip-rap must be able to withstand the repeated mechanical abrasion and wetting and drying action of the waves.

Rocks in the strong to extremely strong range are usually suitable for rip-rap if they can be quarried in intact blocks of sufficient size. Rock types which are commonly used when fresh include:

– quartzite and sandstone;
– limestone, dolomite and marble;
– granite, diorite and gabbro;
– basalt and andesite, and
– gneiss.

Most rocks containing siltstone, shale and claystone would be unsuitable for rip-rap because they would break down (slake) under repeated wetting and drying.

Ideally the rock should meet the durability requirements for concrete aggregates, but many rocks which do not meet these requirements have performed satisfactorily.

Laboratory tests which provide an indication of long term durability include:

– apparent specific gravity and absorption,
– petrographic examination,
– methylene blue absorption,
– accelerated weathering test such as wetting and drying, or sulphate soundness.

Fookes & Poole (1981) discuss the durability requirements for rip-rap.

Rip-rap is commonly obtained from rockfill quarries simply by stockpiling oversized rock from each shot. If a special quarry is required for rip-rap, selection of potential sites must be made bearing in mind that the spacing of persistent joints in the rock mass must be appreciably greater than the required dimensions of the required blocks. It is possible to get a good initial indication of the likely sizes and durability of blocks obtainable from an outcropping source, by observing the joint spacing in the outcrops and the size and condition of surface boulders and/or scree derived from them.

At the site, or sites, selected for detailed exploration, a special objective of the exploratory programs should be determination of the pattern of joints in the rock mass, ie. the number of sets of joints and the orientation, persistence and spacing of joints in each set. In this regard, geotechnical mapping of surface rock outcrops and trench exposures (even if extremely weathered) is important, as it is usually very difficult to determine the joint pattern at depth from drill cores.

During core logging, particular attention should be given to:

– The location of any zones in which the rock appears to be affected by chemical alteration (e.g. in granitic and basaltic rocks – see Chapter 3, Sections 3.1 and 3.2). Such zones often contain minerals which weather rapidly on exposure.

– The distinction between minor joints which may extend for less than 500 mm, and major joints which extend for many metres, and

– The mineral type(s) which form the joint cements, and the apparent strength of the joints. In general, only quartz-cemented joints are likely to be strong enough to resist parting during quarrying and later when blocks are in service.

After logging and photography and taking of samples for strength and durability testing, it is useful to store the drill cores out in the open, in a secure compound. Here, they can be examined and photographed at regular intervals to record any deterioration. It is also possible to accelerate the 'weathering' processes by a program of wetting and drying.

13.4.1.5 *Design of filters under rip-rap*

Where a thin layer of rip-rap consisting of large stones is placed on earthfill, it will be necessary to provide a filter layer between the rip-rap and the earthfill to control erosion of the earthfill from under the rip-rap. In some cases two layers of filter may be required.

Sherard et al. (1963) and US Corps of Engineers (1984a), indicate that the filter requirements between the filter and the underlying soil should be the same as for the requirements for ordinary (Zone 2A) filters and quote the USBR design rule: D_{15} filter $\ngtr 5\, D_{85}$ soil. Sherard et al. (1963) indicate that, for consideration of the filter compared to the rip-rap, it has been proven that provided D_{15} rip-rap $\ngtr 10\, D_{85}$ filter, performance is satisfactory, provided that D15 filter > 50 mm. For most rip-rap, the latter requirement would ensure that the more conservative rule

Table 13.6. Minimum thicknesses of filters under rip-rap (from Sherard et al. 1963).

Wave height (m)	Minimum filter thickness (mm)
0-1.2	150
1.2-2.4	225
2.4-3.0	300

sugested for design of rip-rap versus filter recommended by US Corps of Engineers (1984), i.e. D_{15} rip-rap $\not> 5$ D_{85} filter would be met.

The authors' view is that the design of these Zone 2D filters (Figure 13.7a and c) is not as critical as for Zone 2A filters, in that if damage does occur, it can usually be repaired, and that as a result a relaxation of the strict $D_{15} \not> 5$ D_{85} rule is appropriate. If there is a reasonably well graded sandy gravel/gravelly sand from 75 μm to 50 or 75 mm available either naturally or with a minimum of processing, this should be satisfactory in most cases. This is also the experience of Sherard et al. (1963). Filters which consist only of fine or medium sand are unlikely to be satisfactory as they will not self filter against the rip-rap.

US Corps of Engineers (1984a) recommend that filter layers should be at least three 50 percent size stones thick, but not less than 0.23 m.

Sherard et al. (1963) recommend that the thickness be selected after consideration of:

– Size of wave.

– Gradation of the rip-rap. Rip-rap with less fines needs thicker filters.

– Plasticity and erodibility of the embankment earthfill. If the earthfill is well graded non erodible granular soil, it needs less protection than say a fine silty sand or dispersive soil.

– The cost of the filter.

They recommend the minimum thicknesses shown in Table 13.6.

They indicate these should be absolute minimum thicknesses and that if two layers of filters are needed, each should be at least 150 mm thick.

13.4.1.6 *Use of soil cement and shotcrete for upstream slope protection*

In some locations rock suitable for rip-rap may not be available within economic haul distance, and consideration may be given to the use of soil-cement as an upstream facing. Many dams in the USA have been constructed in this way.

ICOLD Bulletin 54: Soil-Cement for Embankment Dams (ICOLD 1986d) describes the use of soil-cement in some detail. The important features of the method are:

The soil-cement is usually placed in horizontal layers as shown in Figure 13.8. The effective thickness normal to the slope has varied between 0.46 and 0.76 m with 0.6 m being the most common. This gives a 2.4 m horizontal layer width for a 3H:1V upstream slope, allowing placement by trucks. Conveyor placement can be used to allow 0.46 m thickness, and on slopes 3H:1V or flatter, compaction up and down the slope can allow use of thinner covers. This may be acceptable for smaller dams.

– Portland cement is mixed with the available soil, conditioned to around optimum water content (standard compaction) and the soil-cement compacted in 150 to 300 mm thick layers to a density ratio of 98%. The soil cement is then cured under water spray.

– A wide range of soils can be used but usually clayey sand and/or silty sand with 10 to 25% passing 75 μm. The plasticity index is usually less than 8.

– The required cement content is determined by laboratory tests, which include wet-dry and

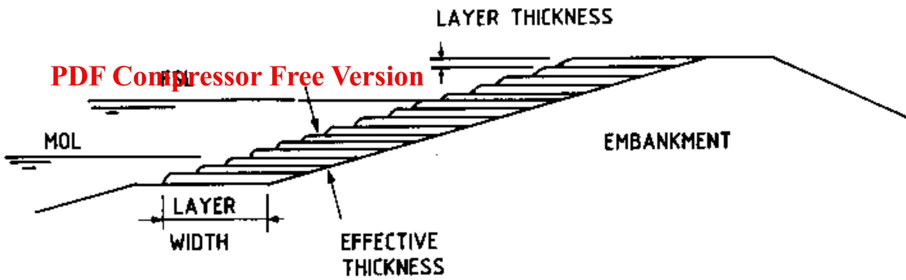


Figure 13.8. Use of soil-cement for upstream slope protection.

freeze-thaw durability and unconfined compression strength. The requirements are detailed in ICOLD (1986d). Freeze-thaw testing would not seem appropriate in many countries with temperate or tropical climates. The USBR require an unconfined compressive strength of 4.1 MPa at 7 days and 6MPa at 28 days. For the field application, 2% cement (by weight) additional to that determined in the laboratory is used to allow for difficulties in mixing and compaction.

Soil-cement has proven to be a satisfactory slope protection over long periods – up to 30 years at least. Readers are directed to ICOLD (1986d) for additional details on the method.

In areas where high quality durable rock in the sizes needed is not readily available, it is possible to provide upstream face protection using an apron of reinforced shotcrete. This was done at Blue Rock Dam near Melbourne, Australia.

Figure 13.9 shows the layout used for Blue Rock. The 200 mm design thickness of reinforced shotcrete was placed on the upper 15 of the 70 m high earth and rockfill dam. The average thickness placed was 270 mm, the additional concrete being used to fill voids between rocks on the embankment and to round off at the toe and the top of the batters. Drainage pipes were provided at the corner of each 6 × 24 m sheet of reinforcement mesh. These were 75 mm PVC sloping 15° out of the slope, and were provided to ensure that water pressure did not build up behind the shotcrete. The steel mesh was placed with a gap of 150 mm between sheets.

13.4.2 *Downstream slope protection*

13.4.2.1 *General requirements*

The downstream slopes of earth and rockfill, and concrete face rockfill dams have a rockfill zone on the downstream slope. For these dams erosion of the face is not an issue and the requirement usually is simply to provide a uniform surface within the tolerance specified. It is not unusual for contractors to go to some trouble to sort larger rocks to the downstream face to give a pleasing appearance to the dam.

For earthfill dams, the downstream face is potentially erodible and considerable care needs to be taken to prevent erosion. This is done by:

– covering the surface with a layer of rockfill, or by establishing grass cover,

– providing berms to limit the vertical distance over which runoff can concentrate,

– providing lined drains on the berms to catch the runoff and carry it to the abutments. Open lined drains may be provided at the contact between the abutment and the dam as shown in Figure 13.10B, or the drains on the berms may be extended along the abutments as shown in Figure 13.10A. In both cases erosion at the contact between the embankment and the abutment is being controlled.

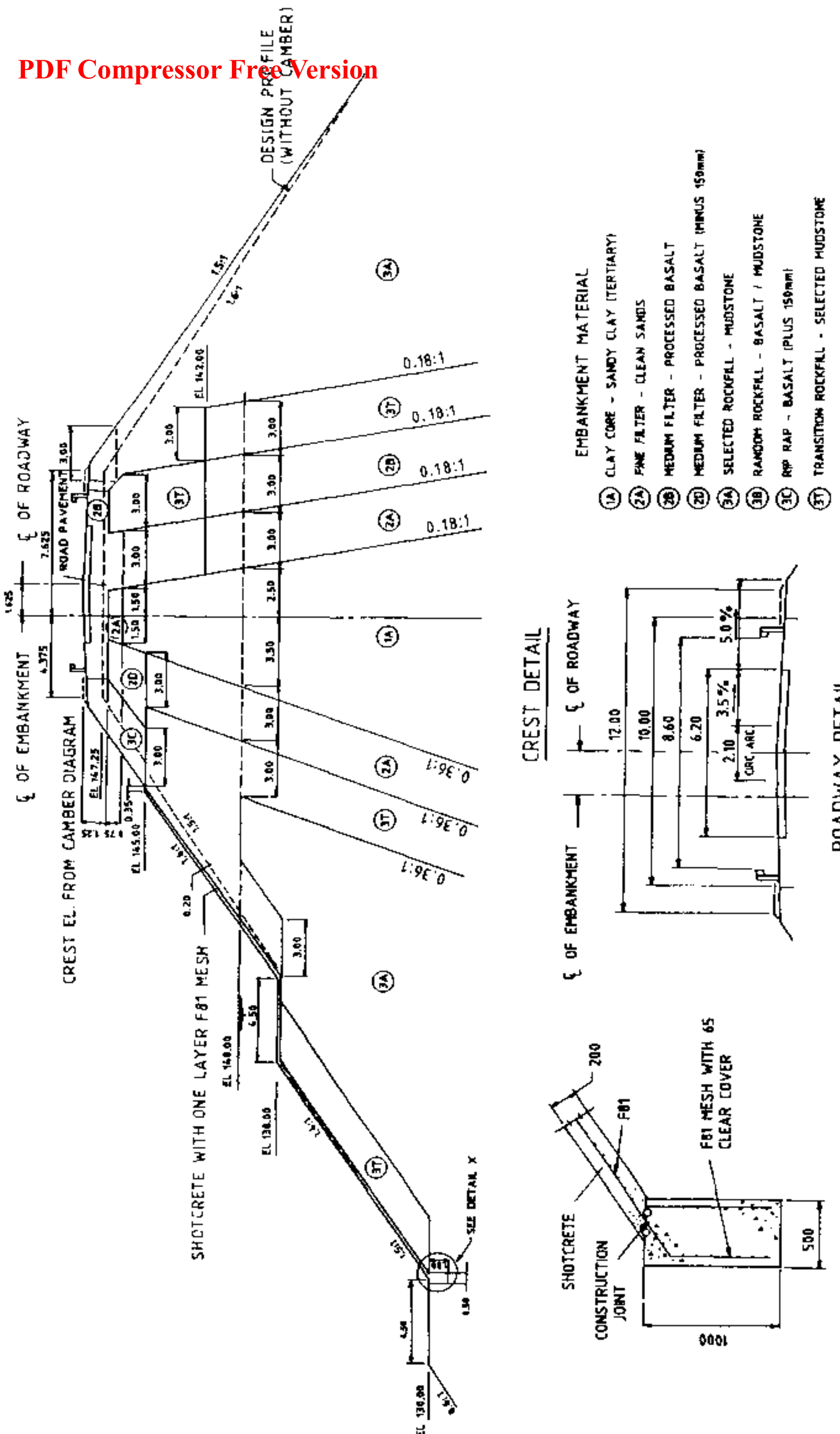


Figure 13.9. Use of shotcrete for upstream slope protection at Blue Rock Dam (courtesy of Rural Water Commission of Victoria).

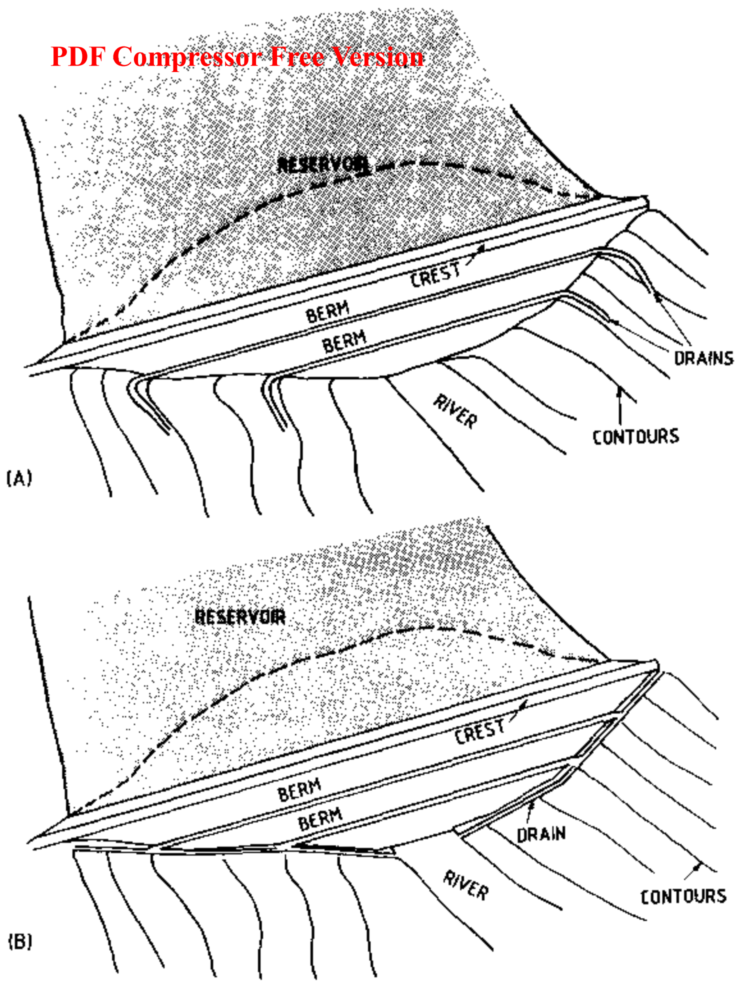


Figure 13.10. Berms and drainage for the downstream slope of earthfill dams where grassing is used to control erosion.

The authors' experience is that berms should be provided at no greater than 10 m intervals vertically. However, in arid climates this may be unnecessary. A berm should be provided above the outlet of a horizontal drain as shown in Figures 13.7a and c. This is necessary to prevent blockage of the outlet to the drain by soil eroded off the embankment.

When first constructed, drains will often block with eroded soil before the grass is well established, and must be inspected and cleaned out regularly. Even when grass is established, maintenance of drains is necessary to ensure they continue to function properly.

13.4.2.2 *Grass and rockfill cover*

The type of grass to be used is dependent on local conditions, particularly the climate and soil, and advice should be sought from local authorities such as the Soil Conservation Service. It is common procedure to provide a layer of topsoil and then to seed the slope using a bitumen hydro mulch which provides initial protection against erosion before the grass establishes. Low native bushes have also been successfully used. It must be expected that the grass will need to be watered at least until it is well established, and that reseeding and repairs will be necessary. USBR (1977) gives a sample specification for establishment of grass cover.

Where there is an ample supply of rockfill and/or climatic conditions preclude the use of grass, dumped quarry run rockfill, placed directly onto the earthfill is usually a satisfactory way of controlling erosion on the downstream slope. USBR (1977) indicate that 0.3 m of rockfill usually is adequate although 0.6 m is usually easier to place.

13.5 CONDUITS THROUGH EMBANKMENTS

The placement of conduits or outlet pipes through earth dam embankments is a common cause of piping failure, particularly in small dams. The problem is particularly acute in dams constructed using dispersive soil. The 'traditional' approach has been to provide concrete cutoff collars around the conduit to lengthen the seepage path.

USBR (1977) recommend that cutoff collars should be constructed of reinforced concrete, generally from 0.6 to 0.9 m high, 0.3 to 0.45 m wide, and spaced from 7 to 10 times their height along the portion of the conduit which lies in the impervious portion of the dam. A bituminous, or other joint filler should be provided between the collar and the conduit to minimise stress concentration effects.

Despite such measures, failures have continued to occur.

It is the authors' experience that inadequate compaction of earthfill in the vicinity of the conduit due to the need to use small hand held equipment, rather than rollers, coupled with compacting dry of standard optimum water content is a significant contributing factor. This has also been noted by Sherard (1973), Sherard & Dunnigan (1985) and others. The authors have also noted that in dispersive soils, cracking of the soil on the sides of a trench into which the conduit was placed, and use of corrugated surface for cutoff collars (to notionally increase seepage path length) have also led to piping failures. These latter failures have occurred despite apparently adequate engineering supervision of construction. Figure 13.11 shows these effects.

The problem is discussed by Sherard (1973) who indicates that in a large percentage of cases, differential settlement around the conduit may cause cracks in the soil, leading to the piping failure.

Sherard (1973) concluded that where conduits must be placed through earth dams:

'1. It is particularly important that the embankment adjacent to the conduit be placed at a relatively high water content and not be a soil susceptible to piping.

2. Even in small, homogeneous dams where no chimney drain is installed it is advisable to provide a drain and filter around the conduit at its downstream end for the purpose of intercepting concentrated leaks which follow the conduit.

3. In cases where the soil foundation is thick and compressible, it is not desirable to excavate a trench under the conduit and fill it with compacted earth.'

Sherard & Dunnigan (1985) indicate that because of concerns relating to compaction of soil adjacent cutoff collars, and the increased confidence in the use of filters to control erosion, they recommend that:

– no seepage collars be provided;

– concrete surfaces surrounding conduits should be smooth and the sides should be sloped at 1H:8V or 1H:10V so that earthfill can be compacted directly against the concrete;

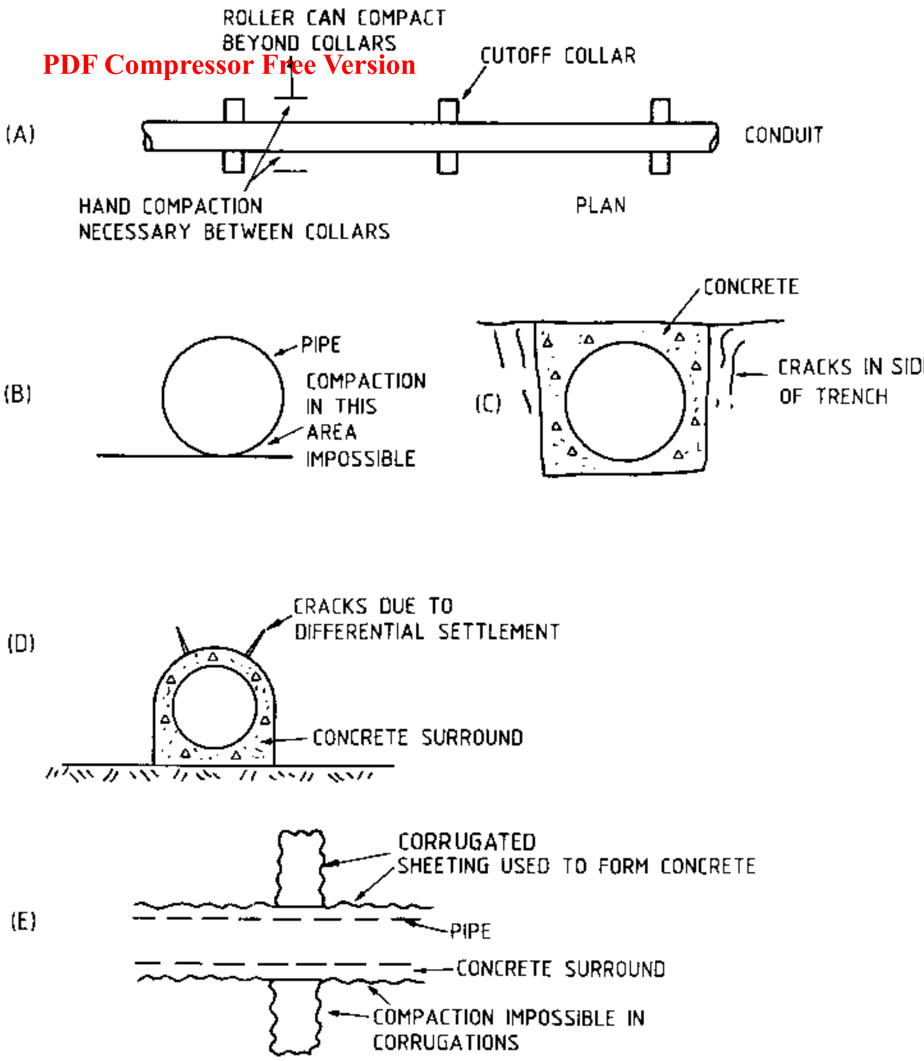


Figure 13.11. Some causes of piping failure around conduits: (A) Inadequate compaction due to the presence of cutoff collars; (B) Inadequate compaction under pipe; (C) Cracking in soil or extremely weathered rock in the sides of a trench; (D) Cracks due to differential settlement; (E) Use of corrugations or other roughening of the surface of cutoff collars or concrete surround.

– a filter be provided to surround the downstream position of the conduit, ie. underneath as well as on both sides and the top so that all potential leakage travelling along the concrete-earth core interface exits in a controlled manner.

They indicate that this is common practice.

Talbot & Ralston (1985) indicate that the Soil Conservation Service of the USA have replaced conduit cutoff collars with a filter diaphragm as shown in Figure 13.12.

They indicate that a single diaphragm is constructed around outlet conduits or other structures in the downstream section of the dam as shown in Figure 13.12. The diaphragm consists

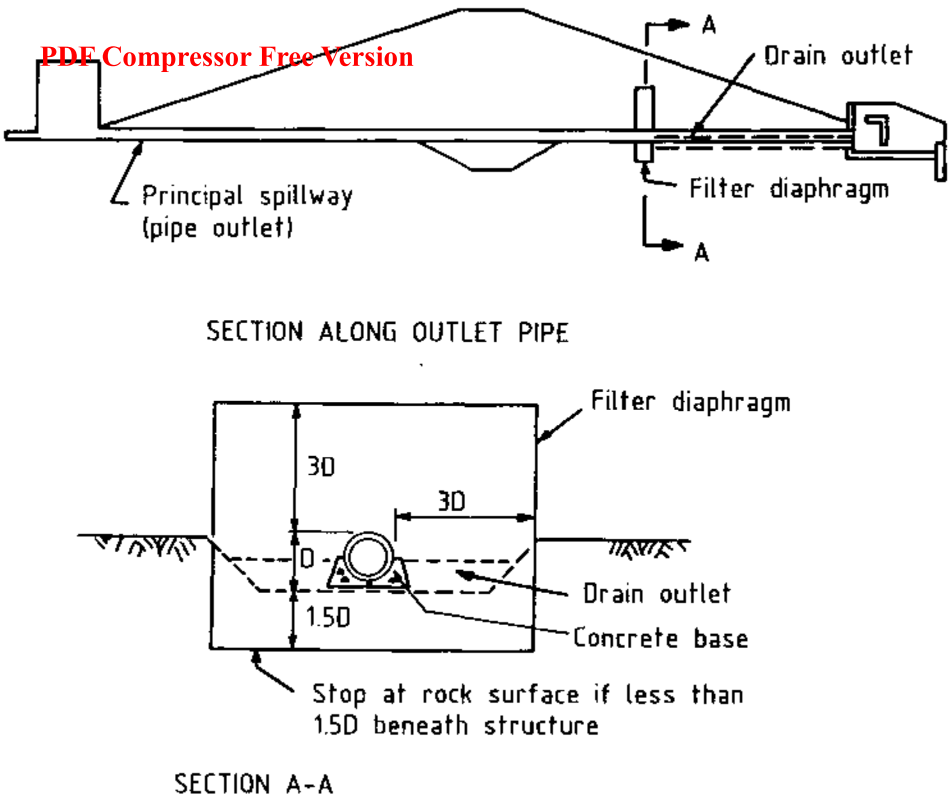


Figure 13.12. Filter diaphragm for seepage and piping control around outlet pipe (Talbot & Ralston 1985).

of a graded sand-gravel filter that projects outward a minimum of 3 times its diameter (for pipes) except in a downward direction where differential settlement is not anticipated (i.e. rock). Where other embankment or foundation drainage systems are used, the diaphragm must tie into these systems to provide a continuous zone that intercepts all areas subject to cracking, poor compaction, or other anomalies.

It is the authors' opinion that such an approach is entirely logical in view of the fact that some seepage along the concrete-earthfill interface is inevitable. This, coupled with a proper concrete surround, or base as shown in Figures 13.11d and 13.12, and, compaction at or above optimum water content to a density ratio of 98% is recommended. No concrete cutoff collars should be provided. If the soils are highly dispersive (e.g. Emerson class 1 or 2) in the vicinity of the outlet conduit, it is also recommended that the soil is modified by addition of 2 to 3% lime so as to render it non dispersive. This would be essential in dispersive soils where a seepage diaphragm is not provided.

The overriding consideration should be to try and avoid placing the outlet conduit through the embankment or in soil under the embankment, since despite the precautions outlined above, the potential for piping along the conduit will remain.

Care must also be taken to ensure that joints in the outlet conduit cannot open and allow soil to wash into the conduit.

13.6 INTERFACE BETWEEN EARTHFILL AND CONCRETE STRUCTURES

Just as the contact between earthfill and outlet conduits can be a potential zone of weakness in an embankment, the contact between earthfill and concrete structures, e.g. the wall of a concrete spillway, can be potential sources of cracking and piping failure.

Earlier practice was to construct a cutoff wall at right angles to the wall of the structure to penetrate into the earthfill and increase the seepage path. Figure 13.13 shows an example of this, and the difficulties that are created because rollers cannot be used to compact the soil adjacent the walls.

To overcome this Sherard & Dunnigan (1985) recommend that no cutoff wall be provided and that the contact be detailed as shown in Figure 13.14. This approach is recommended by the authors as again recognising the inevitability of seepage and possible cracking, and seeking to control such seepage by well designed filters.

For this arrangement, compaction of earthfill above standard optimum water content with rubber tyred equipment is possible right up to the wall. The wall is left smooth, off form concrete, not sandblasted, chipped, painted or treated in any way. The downstream filters are widened. Sherard & Dunnigan (1985) suggest that if the Zone 2A filter contains a significant quantity of gravel sized particles, there is a danger the gravels could segregate at the concrete-filter interface leaving pockets of gravel without sand in the voids. In this situation they suggest placing an additional filter consisting only of sand upstream of the Zone 2A filter (as well as widening the Zone 2A and 2B filters).

They suggest that the angle at which the dam axis intersects the wall (angle α in Fig. 13.14) may be made say 80 to increase the contact pressure on the contact as the dam water load is

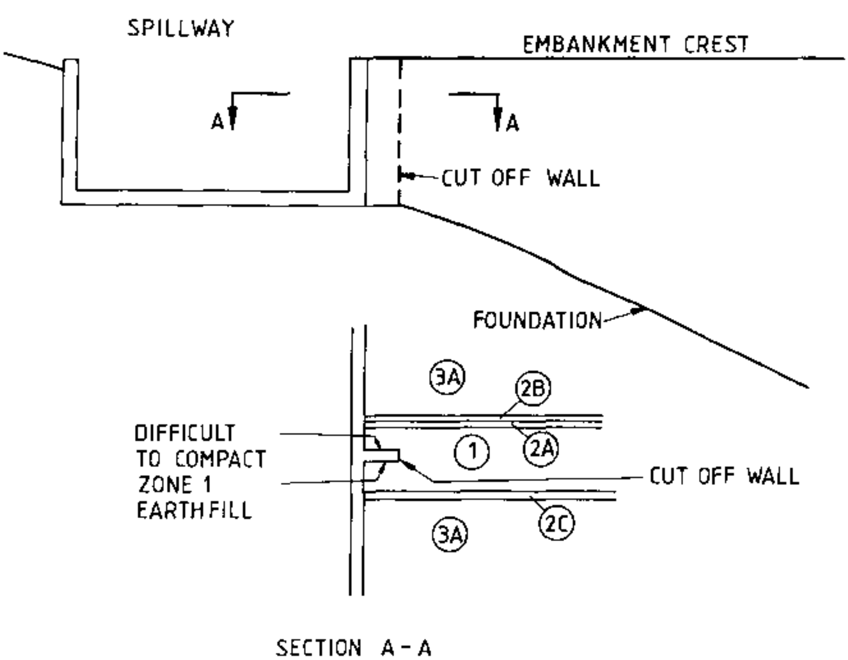


Figure 13.13. Contact between earthfill and spillway wall with a cutoff wall provided.

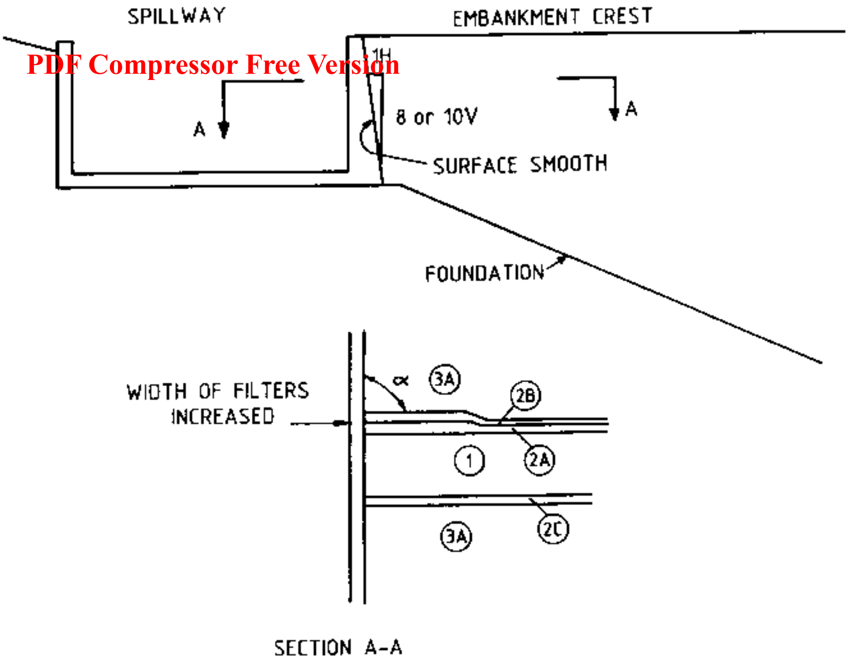


Figure 13.14. Detail for interface between earthfill and concrete structures.

applied. They point out, however, that many dams have been successfully constructed with $\alpha = 90$.

It is recommended that if highly dispersive soils are being used for construction, consideration should be given to adding lime in the vicinity of the contact with the concrete structure (in addition to the features detailed above).

Where the dam generally has no filters, consideration should be given to providing a filter in the vicinity of the contact.

13.7 FLOOD CONTROL STRUCTURES

Flood control structures, e.g. flood storage basins and levee banks, have particular features which require consideration. These include:

– the embankments stand without water adjacent them for long periods;
– water levels rise quickly, and usually fall within days or even hours;
– the consequences of failure may not be as critical as for other dams (but not always);
– the embankments may serve other purposes, e.g. road, parking area.

From a design viewpoint this has the following implications:

a) The embankments may crack due to desiccation if built of clay soils (as they usually are).

b) If the soils are dispersive, the cracks may lead to piping failure as the water levels rise quickly. Hence for dispersive soils, erosion control must be provided by (i) modifying with lime or gypsum to make the soil non dispersive; or (ii) providing filters to control erosion.

That is, the embankment must be treated with just as much caution as a normal dam (if not more).

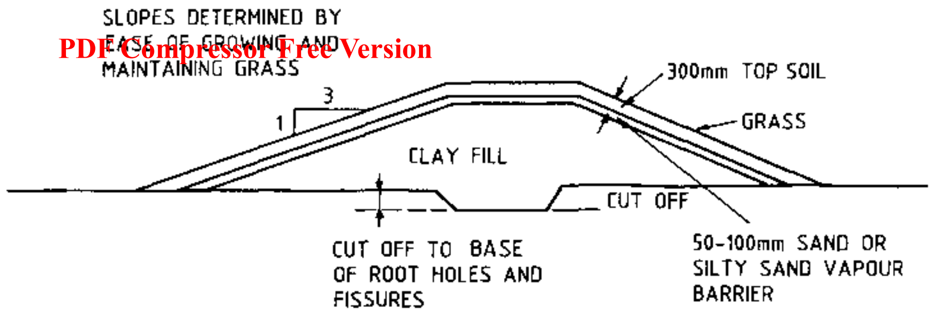


Figure 13.15. Possible detail for flood control structure.

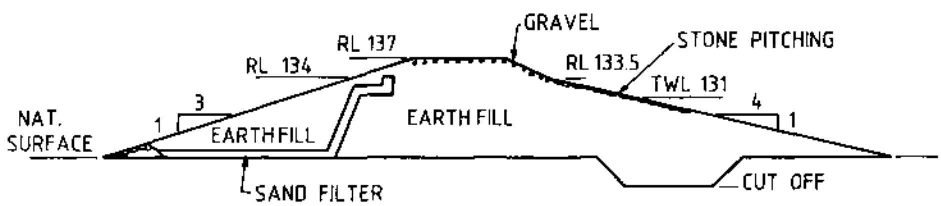


Figure 13.16. Levee bank for Fitzroy River, Western Australia (Moulds 1987).

Cracking can be reduced by providing a vapour barrier as shown in Figure 13.15.

The vapour barrier consists of a thin layer of silty sand which prevents high suctions developing in the clay. This should be sufficient for low hazard structures built of non dispersive clays, i.e. filters are not needed in the design. An alternative would be to place a layer of geotextile fabric to act as the vapour barrier. A road pavement on the crest would also assist in preventing cracking.

Note that a cutoff should still be provided although possibly not to the same standard as for a normal dam.

c) The embankments are unlikely to reach the steady state seepage condition (or instantaneous drawdown), so slopes can be steepened compared to what would be required for a normal dam.

Hence from slope stability considerations, embankments built of clay soils can stand at 1.5H to 1 V. Silty sands, or sandy soil embankments will, however, saturate quickly and require flatter slopes.

From the erosion control viewpoint, slopes of 3H to 1 V or flatter are preferable, and this may override stability considerations.

The partial saturation effect may be more important for levee banks on the side of a river bank if it can be shown that the bank will not saturate. However, care must be taken as most bank failures occur on drawdown of the flood. An example of a levee bank on the Fitzroy River, Western Australia, is given in Figure 13.16.

13.8 DESIGN OF DAMS FOR OVERTOPPING DURING CONSTRUCTION

13.8.1 *General design concepts*

When embankment dams are constructed on a river it is necessary to divert the river from the river bed during construction of the dam. This is usually done by constructing a diversion tunnel through an abutment of the dam, and constructing a coffer dam upstream of the embankment to divert the river into the diversion tunnel. This upstream coffer dam may be supplemented by a higher coffer dam which is incorporated into the main embankment as shown in Figure 13.17.

When designing coffer dams a balance is reached between the height and therefore cost of the coffer dam, and the recurrence period of the flood which will overtop the dam. Also important are the consequences of overtopping, in potential damage to the construction works, delay to construction schedules, and potential for damage or loss of life downstream. Of particular concern is the situation if the coffer dam fails, giving a substantial flood wave.

In the example shown in Figure 13.17, the upstream coffer dam has been designed to divert the 1 year summer (dry season) flood, the main coffer dam the 5 year winter (wet season) flood, and there is, therefore, some risk that it will be overtopped.

The practice in Australia since the late 1960's has been to provide protection to the coffer dam and/or to the main embankment to prevent failure in the event of overtopping. This is done by covering the downstream face of the coffer dam or the main embankment, with steel mesh or rock filled gabions, which are anchored into the slope with steel reinforcing rods. If this reinforcement is provided on the main embankment it can be designed so that much greater return period floods can be accommodated, e.g. 1 in 100 or 1 in 700 year floods in Figure 13.17.

The use of such steel mesh reinforcement has allowed use of significantly smaller diversion tunnels and upstream coffer dams, with resultant savings in project cost. The design construction, and operation of steel mesh reinforcement is described in ANCOLD (1982) and ICOLD (1984). As detailed there, up to 1982, the method had been used on 50 dams, 4 in Australia, 9 in other countries.

The following outlines only the broad principles, and reference to the ANCOLD or ICOLD publications, and references therein is recommended for design purposes.

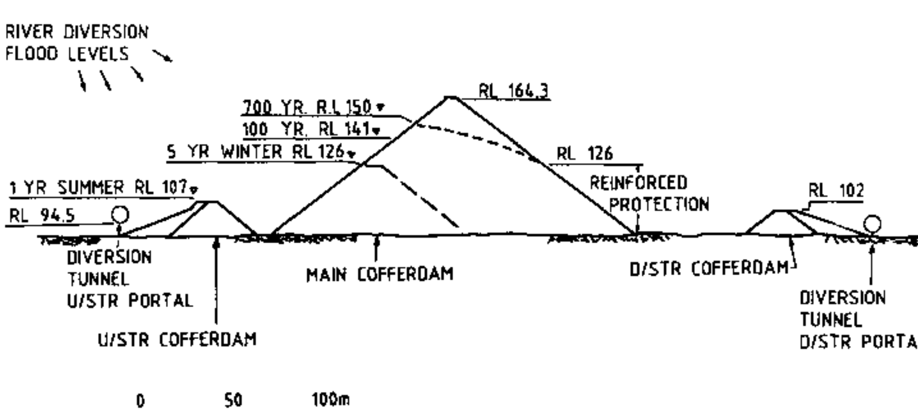


Figure 13.17. Coffer dams for river diversion.

13.8.2 *Types of steel mesh reinforcement*

The technique has evolved over several years, being modified to take account of lessons learnt from failures, and to reduce the costs involved. Some early systems failed during operation, due to:

– Damage to relatively lightweight steel meshing, by floating logs, or by boulders from the surface of the dam, e.g. Cethana Dam (HEC 1969).

– Erosion of the contact between the steel meshing and the downstream abutment due to inadequate anchorage (e.g. Xonxa Dam, Pells 1978).

A design which took account of these early problems, and successfully withstood 33 hours of overtopping, up to 2.5 m deep when the dam was 20 m high, was that employed at Googong Dam (Fokkema et al. 1977).

Figure 13.18 shows the meshing system for Googong Dam.

There are several features of the design:

– the cranked anchor system for the avoidance of progressive failure,

– the combination of a medium duty surface mesh with small opening size and an overlay of fairly closely spaced (500 mm centres) sloping bars to protect the mesh,

– the return of the sloping bar into the fill for tensioning to the anchor prior to welding the connection between the two,

– the continuous concrete protection around the toe.

The Hydro-Electric Commission of Tasmania have done a significant amount of developmental work in steel mesh reinforcement. Figures 13.19, 13.20 and 13.21 show systems they have developed to cope with their particular problems – regular overtopping, particularly of coffer dams, and heavily timbered catchments, which give potential for damage during overtopping by floating logs.

These all use cylindrical gabions fabricated from 50 mm square mesh with 5 mm wires both ways (4 mm for Mackintosh Coffer Dam).

For the Mackintosh Coffer Dam, a concrete slab was placed over the rockfill crest to secure the top row of gabions and prevent overflowing water from entering the downstream rockfill zone through the crest. Model tests were carried out to measure the pore pressures used in stability analysis which showed that anchor bars were not required.

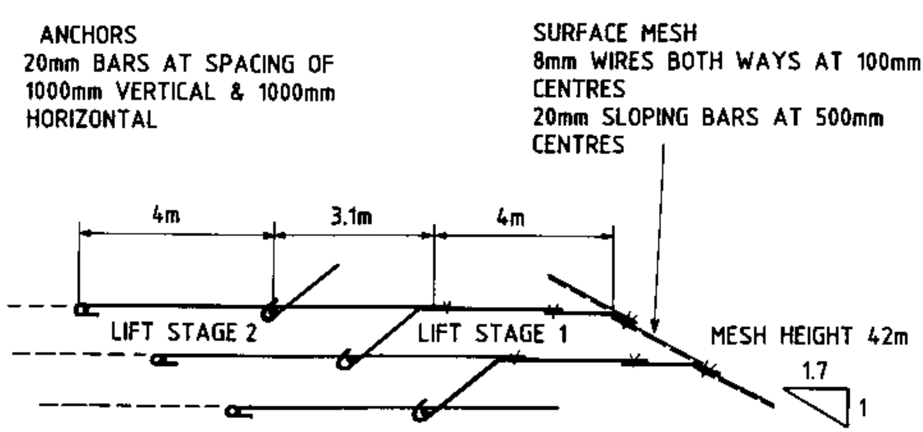


Figure 13.18. Steel mesh reinforcement for Googong Dam (Fokkema et al. 1977 and ANCOLD 1982).

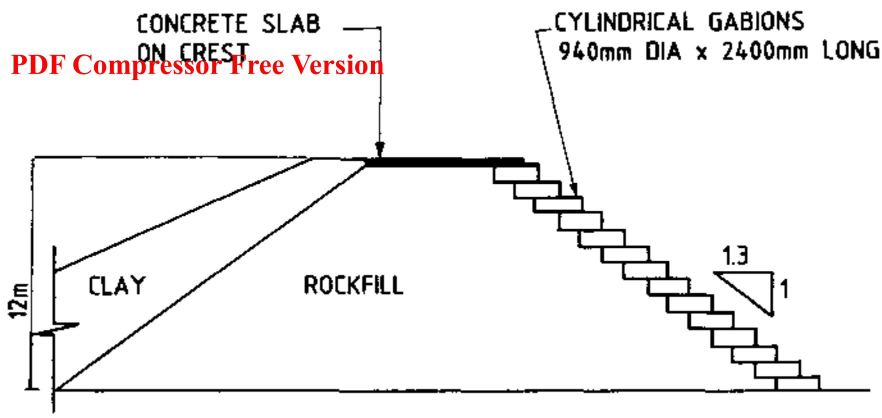


Figure 13.19. Steel mesh reinforcement for Mackintosh Coffer Dam (ANCOLD 1982).

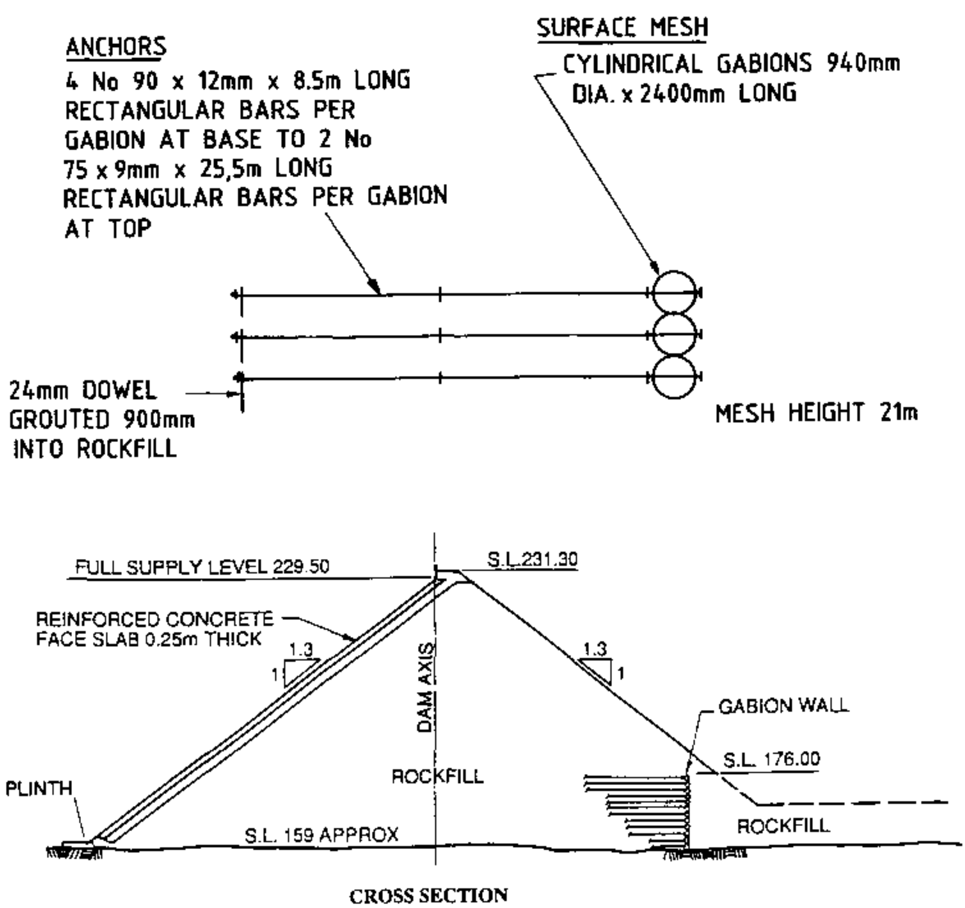


Figure 13.20. Steel mesh reinforcement for Mackintosh Dam (ANCOLD 1982).

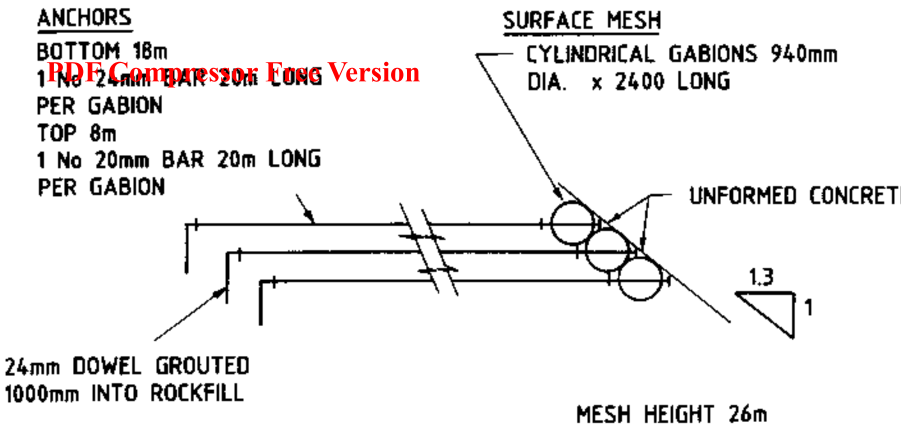


Figure 13.21. Steel mesh reinforcement for Murchison Dam (ANCOLD 1982).

The coffer dam was overtopped by small floods several times without any damage being sustained. A similar system has been used on other HEC dams.

For Mackintosh Dam the downstream face of the dam was protected to a height of 21 m. As shown in Figure 13.20 this took the form of a vertical wall of gabions anchored to dowels grouted into the rockfill. The protection was made vertical to keep the many floating logs that were expected to be carried during floods, clear of the face. The triangular wedge of rockfill forming the toe of the dam was placed after the embankment was above the 100 year flood level.

The dam was overtopped once during construction but only to a depth of 300 mm.

The system was found to be costly mainly because of the very substantial anchors required and the full strength bolted connections. Although this type of connection was used in preference to a welded connection to avoid delays in wet weather, the large number of anchors and connections slowed down the installation of the system and delayed embankment construction.

For Murchison Dam, the gabions were placed on the same slope as the face of the dam as shown in Figure 13.21. The cylindrical gabions could be installed rapidly and did not control the rate of embankment construction.

The exposed upper surface of the gabions on the dam face was protected with unformed concrete.

There was no overtopping event during construction, although flow through the rockfill occurred in a flood which ponded above the upstream coffer dam when the dam was well above the mesh level.

A similar system has been used on later HEC dams.

A recent application of the Googong type of meshing system is shown in Figures 13.22, 13.23 and 13.24 for Clarrie Hall Dam. This incorporates a rather simpler system of anchorage than the cranked bars used at Googong and pays particular attention to preventing erosion at the abutment of the dam. Figure 13.22b shows the detail in the river, and Figure 13.22c the abutments.

A critical issue in steel mesh reinforcement is to keep the crest level as uniform as possible, and to be able to tie down the upper surface. Figure 13.24b shows gabions which were used in the event of imminent overtopping in lower parts of the embankment crest.

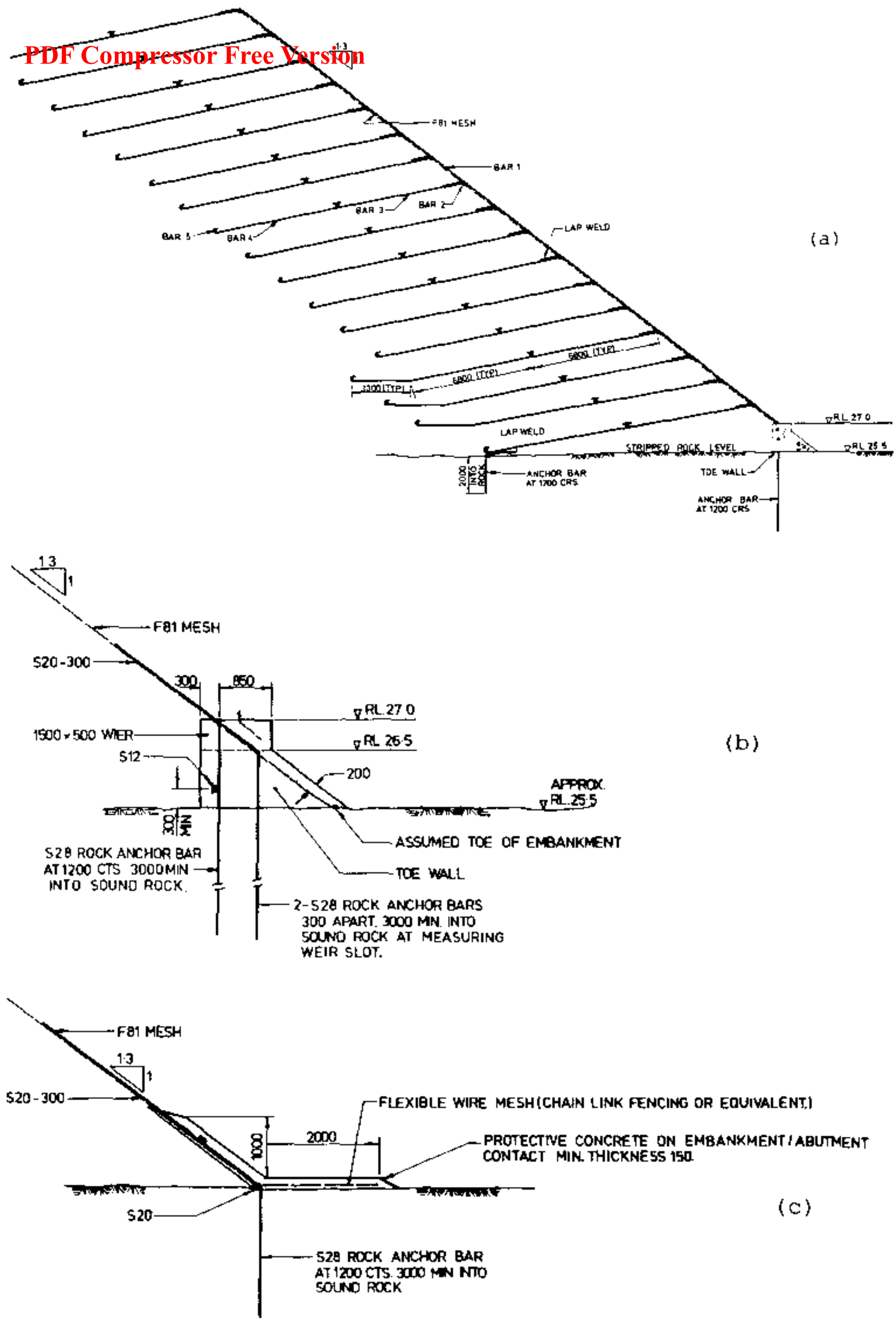
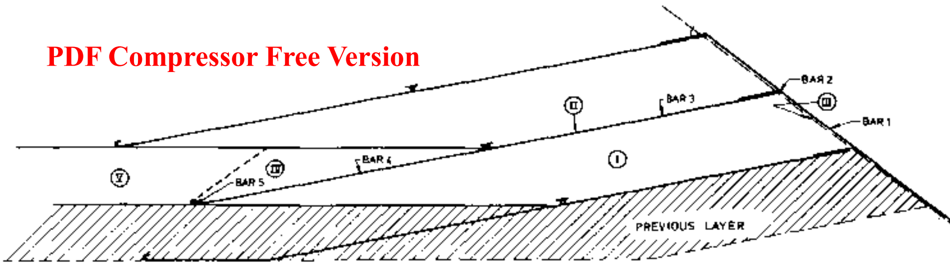


Figure 13.22. Steel mesh protection for Clarrie Hall Dam, a) meshing layout; b) abutment detail, river section; c) abutment detail, other than river sections (PWD of NSW 1981).



CONSTRUCTION SEQUENCE DIAGRAM

NOTES

CONSTRUCTION SEQUENCE

1 BEFORE COMMENCEMENT OF ANY MESHING PREPARE AND HAVE ON HAND 24 GABIONS 2·0 m LONG × 1 m WIDE × 0·5 m DEEP

2 CONSTRUCT ROCKFILL WINDROW (I) COMPACT AND TRIM FACE

3 CONSTRUCTION ON WINDROW SHALL NOT PROCEED MORE THAN ONE SHIFT IN ADVANCE OF MESH PLACEMENT

4 PLACE AND TIE F81 FACE MESH (III) PLACE TIE BARS (II) (BARS 3 AND BAR 4 AND ANCHOR BAR 5)

5. PLACE ROCKFILL (IV) OVER BAR 4 AND PROCEED WITH PLACEMENT AND COMPACTION OF LAYER (V) PLACE FACE COVER BAR (BAR 1) AND HORIZONTAL BAR (BAR 2) AND WELD COVER BAR (BAR 1) TO PREVIOUS LAYERS

6. WELD BAR 1 TO BAR 2.

7. BEND BAR 1 TO ALIGN WITH BAR 3. TENSION BAR 3 TO 5 kN AGAINST BAR 1 AND LAP WELD.

8. IN EVENT OF A THREATENED OVERTOPPING, DO NOT ADVANCE WINDROW FURTHER. COMPLETE TRIMMING AND MESHING INCLUDING TENSIONING AND WELDING OVER EXPOSED WINDROW. PLACE PROTECTIVE GABIONS ALONG WINDROW EDGE AND WELD TO MESH. TIE GABIONS TOGETHER.

Figure 13.23. Steel mesh reinforcement for Clarrie Hall Dam – construction sequence (PWD of NSW 1981).

13.8.3 *Design of steel reinforcement*

The design principles are:

a) Control ravelling of the rockfill by steel mesh or gabions.

b) Protect the mesh from damage from floating debris by placing reinforcing bars up and down slope over the mesh; constructing the slope very steep with gabions so debris shoots out beyond the mesh/gabions; or cover (in part) with concrete.

c) Prevent damage to the mesh from rockfill washing off the top of the dam by ensuring that the meshed section is always the highest point on the dam section. Rapid 'closure' methods are needed in the event of flooding being predicted.

d) Anchoring the mesh into the dam in such a way that the anchors are always protected from erosion by the next layer of fill) i.e.crank-shaped anchors (Googong); inclined anchors (Clarrie Hall); anchors fixed to grouted dowells (Mackintosh).

e) Anchor length is determined by slip circle or wedge analysis, with anchor forces included. Pore pressures are estimated from design charts (see Fitzpatrick 1977 and Lawson 1987), or preferably by hydraulic model tests.

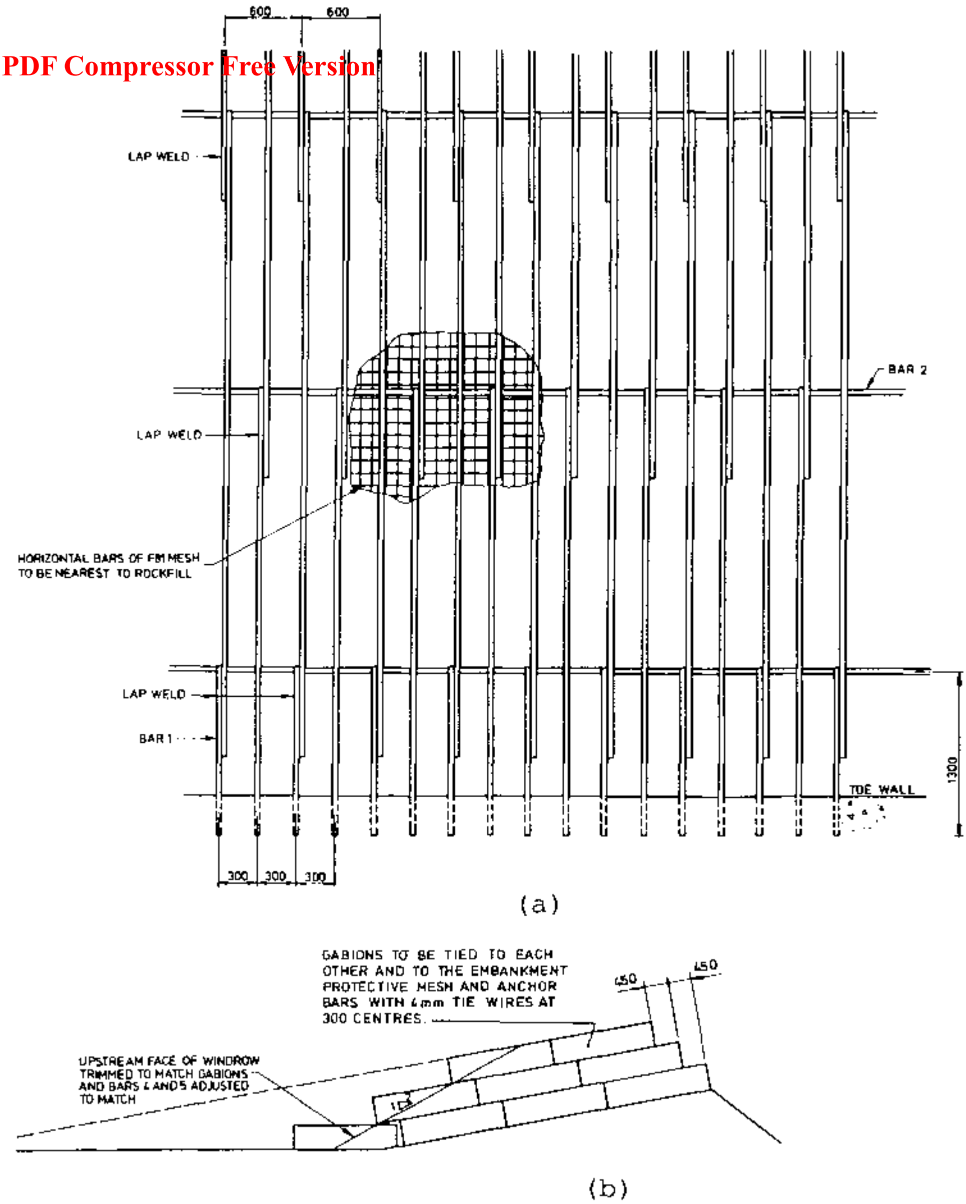


Figure 13.24. Steel mesh reinforcement for Clarrie Hall Dam, a) dam detail; b) crest gabions.

f) The mesh is anchored into the abutment to prevent it being eroded out. Flow concentrates in this area during overtopping. Concrete protection may be provided (as in Clarrie Hall).

Detailing is vitally important and reference should be made to the papers referenced in ANCOLD (1982) and ICOLD (1984).

CHAPTER 14

Specification and quality control of earthfill and rockfill

14.1 SPECIFICATION OF ROCKFILL

It is common to specify the following for rockfill for dam construction:

a) The source, type of rock and degree of weathering, e.g. 'slightly weathered to fresh granodiorite from the spillway excavation.'

b) The maximum particle size and layer thickness. These are usually the same. The layer thickness (after compaction) would normally be specified as: 1.0 m for Zone 3A; 1.5 to 2 m for Zone 3B; except that adjacent to the Zone 2B filter, Zone 3A may be placed in 0.5 m thick layers to give a gradation from Zone 2B to Zone 3A. If the rock is susceptible to breakdown under rolling due to its nature, e.g. a sandstone, and/or settlement is important (e.g. for a concrete face rockfill dam) the layer thickness everywhere for Zone 3A may be reduced to around 0.5 m. Table 14.1 gives examples of requirements for some Australian dams. Table 14.2 gives more properties of the rockfill for some of these dams.

c) The rockfill particle size is further specified to ensure some medium size rock fragments in the rockfill, and to limit the amount of 'fines.' Examples are given in Table 14.3. Care needs to be adopted in specifying such requirements, particularly for small dam projects as:

– It is costly and difficult to carry out the testing to determine the particle size in the embankment. For rock with a maximum size of 1 m, a hole about 4 to 5 m diameter, and to the depth of the layer would have to be dug to obtain a representative sample. Several tonnes of rockfill are involved.

– The upper part of each layer is likely to be finer than the lower part, because of breakdown of rockfill under the rolling action. If vertical permeability is critical the permeability of this broken material will dominate. If, however, it is a normal situation where horizontal drainage is most important, such breakdown may not be important provided the lower part of the layer remains permeable.

– For poor quality rockfill, e.g. those from many sandstones and siltstones, such tight standards will not be achievable.

The authors favour an approach which requires:

– the maximum particle size to be the layer thickness,

– a limit on the amount of silt and clay sized 'fines', e.g. the ≯ 10% passing 0.075 mm specified for concrete face rockfill dam,

– water not to pond on the surface of the compacted layer (for free draining rockfill).

d) Roller type and number of passes. It is normal to specify smooth steel drum vibrating rollers for compacting rockfill. The roller is usually specified as having a static mass of between

Table 14.1. Specification of compaction of rockfill and resulting properties – Concrete face rockfill dams.

Authority dam	Rock type and weathering	Zone	Layer thickness/ Max. size (m)	Compaction: Roller	Compaction: Passes	Compaction: Water applied	Rockfill drainage capability	Rockfill modulus	Reference
PWD of NSW									
Mangrove Creek	Sandstone & siltstone, FR	2E transition	0.45/0.45 incr. to 0.6 m	10 TVR	4	15% by vol	Semi-impervious 10^{-4} to 10^{-9} m/sec		McKenzie & McDonald, 1981
	Sandstone & siltstone, FR	2D transition	0.3/0.15	10 TVR	4	15% by vol	Mean 10^{-5} m/sec		
	Siltstone & sandstone, FR	3A 'free draining'	0.6/0.6	10 TVR	4?	15% by vol	'Free draining'		
	Sandstone & siltstone FR & W	3A	0.45/0.45	10 TVR	4?	OWC – 3% to OWC – 1% approx.	Random zone, not free draining	120 MPa	
MMBW									
Winneke	Sandstone, SW	3A	0.9/0.7	10 TVR	4	15% by vol	'Free draining'	50 ± 15 MPa	MMBW, 1981
	Siltstone, FR & claystone MW-HW	3B random	0.5/0.4	10 TVR	6	8% to 11%	Random fill, not free draining		
WRC of NSW									
Glennies Creek	Welded SW-MW	3A	1.0/1.0	10 TVR	4	20% by vol	Free draining	–	Vesk, 1981
	Tuff SW-FR	3A free draining	1.0/1.0	10 TVR	4	20% by vol	Drainage layer		
	SW-MW	3B	2.0/2.0	10 TVR	6	20% by vol	Free draining		
Brogo	Granodiorite	3A	1.0/1.0	8.6 TVR	4	Not reqd.	Free draining		Vesk, 1981
		3B	2.0/2.0	8.6 TVR	6	Not reqd.	Free draining		
Pindari	Porphry	3A	0.9/0.9	8.6 TVR	4	Not reqd.	Free draining		Vesk, 1981
	Rhyolitic tuffs & lavas	3B	1.8/1.8	8.6 TVR	6	Not reqd.	Free draining		
E & WS of SA									
Kangaroo Creek	Schist	3A	0.9/0.9 to 1.8	10 TVR	4	100% by vol	Semi impervious upper part each layer		Good, 1981
Kangaroo Creek	Gneiss, FR-MW	3A free draining	1.22/1.22	10 TVR	4	100% by vol	Free draining		
HEC, Tasmania									
Mackintosh	Greywacke & slate	3A	1.0/1.0	10 TVR	4-8	10% by vol	Free draining	40/95 MPa rock load	Fitzpatrick et al. 1985
		3B	1.5/1.5	10 TVR	4-8	10% by vol	Free draining	Water load 145/310 MPa	
Cethana	Quartzite	3A	0.9/6	10 TVR	4-8	15% by vol	Free draining		
		3B	1.35/0.9	10 TVR	4-8	20% by vol	Free draining	225/650 MPa	

Murchison	Rhyolite	3A	1.0/1.0	10 TVR	8	20% by vol	Free draining	225/650	
	SW, FR	3B	1.5/1.5	10 TVR	8	20% by vol	Free draining	MPa	
Lower Pieman	Dolerite	3A	1.0/1.0	10 TVR	4-8	Added to trucks	Free draining	160 MPa	
		3B	1.5/1.5						
Water Res. C. Qld.									
Boondoomba	Rhyolite	3A	0.75/0.6	10 TVR	4	10% by vol	Free draining		Rogers, 1985
		3B	1.5/0.9	10 TVR	4	10% by vol	Free draining		
S.M.E.C.									
Khao Laem	Limestone	3A	1.0/0.9	10 TVR	4	15% by vol	Free draining	40-50 MPa rock load	Watakeekul, Roberts & Cole, 1985
		3B	2.0/1.5	10 TVR	4	Lower levels only	Free draining	30-40 MPa rock load 165-300 MPa	
PWD of WA									
Harding	Dolerite SW-FR	3A	1.0/1.0	10 TVR	4	15% by mass	Free draining	N/A	Wark & Meinck, 1985
MMBW									
Cardinia Creek	Siltstone, sandstone	3A	1.8 m/1.8 m	8 TVR	4	Not reqd.		N/A	MMBW 1970
Thomson	SW-FR	3A	1.0/1.0	50-55 kN/1 m of roller static	4	Not reqd.		N/A	MMBW 1978
Water Authority of W.A. Harris	Granite SW-FR	3A	1.0	8-12 tonne VR	4	Not reqd.	Free draining	N/A	

Note:
PWD of NSW = Public Works of Department of New South Wales.
MMBW = Melbourne and Metropolitan Board of Works, Victoria.
WRC of NSW = Water Resources Commission of New South Wales.
E & WS of SA = Engineering and Water Supply Department of South Australia.
HEC, Tasmania = Hydro Electric Commission, Tasmania.
Water Res. C. Qld. = Water Resources Commission, Queensland.
S.M.E.C. = Snowy Mountains Engineering Corporation.
PWD of WA = Public Works Department of Western Australia.
Water Authority of W.A. = Water Authority of Western Australia.

Table 14.2. Some properties of rockfill for concrete face rockfill dams.

Dam	Rock type	Weathering grade	Bulk density (t/m^3)	Absorption (%)	Unconfined strength		SSD/OD	Point load strength (MPa)
					Oven dry (MPa)	Saturated surface dry (MPa)		
Mangrove Creek	Sandstone	FR	2.25	5.1	44.9	22.6	0.5	1.48-1.82
		W	2.11	6.7	26.0	10	0.38	0.55-0.74
	Inter. Siltstone & Inter. Siltstone	FR	2.44	3.7	64	29.6	0.46	1.44-2.25
	Sandstone	W	2.53	3.0	55			
	Siltstone	FR	2.55	2.7	44.5	25	0.56	
	Claystone	FR	2.71	2.1	18	12	0.67	
Winneke	Sandstone, siltstone	SW & FR	–	–	30-114, Av. 58			0.9-17, Av. 13
	Claystone	MW-HW			17-42, Av. 24			0.2-9.2, Av. 7
Glennies Creek	Welded tuff	FR	2.57	0.9	325		0.8	
		FR-SW	2.38	3.6	217		0.7	
		SW-MW	2.42	2.8	228		0.6	
		FR-SW	2.35	4.1	184		0.64	
		SW-MW	2.24	5.1	139		0.56	
Brogo	Granodiorite	M-E	2.53-2.03	0.81-1.85	10-42		0.39-0.5	
		FR	2.65-2.7	0.13-0.51	60-127		0.45-0.89	
Pindari	Porphry	FR(1)	2.36-2.42	2.9-3.8	97, 112		0.42, 0.86	
		FR(2)	2.50-2.6	0.7-1.5	100, 183		0.62, 1.02	
		FR(3)	2.4-2.52	1.5-3.4	166		0.81	
		SW(1)	2.28-2.44	2.4-4.8	77-98		0.31-1.31	
		SW(2)	2.46-2.43	1.6-2.1	155, 169		0.14, 0.59	
Kangaroo Creek	Schist Zone 3			3	25			
	Gneiss Zone 1			1.5	78			
Boondooma	Rhyolite	FR-SW	2.23-2.61	3.0-6.3	103-158	50-113	0.42-0.99	

8 and 12 tonnes (usually 10 tonnes) and a centrifugal force not less than say 240 kN 'at the maximum frequency permitted by the manufacturer, for the continuous operation of the roller' (PWD of WA 1982). Alternatively (MMBW 1980, for Thomson Dam) the static weight may be specified as a weight per metre of drum (50 to 55 kN/m), and a centrifugal force/m of drum (125 kN/m). The frequency was also specified (16 to 25 Hz) for Thomson Dam. Reference should be made to manufacturers' specifications for rollers before specifying such figures.

e) Roller trials are often specified to determine the number of passes. These are discussed below.

f) Rockfill is often required to be 'hard' and 'durable.' The means of measuring this are seldom specified and in many dams, such a requirement will be unobtainable, e.g. where siltstone or sandstone is being used.

Many designers also have a requirement for a compacted density for the fill, a common requirement being around 2.1 to 2.2 tonnes/m^3. Alternatively a void ratio (volume void/volume solids) of 15 to 25% is required. The design is seldom sensitive to bulk density and again, for large rockfill, it is difficult and costly to measure density or void ratio because of the large volume which has to be sampled. Hence, it is unlikely density will be checked for smaller dams, and relatively few tests will be done even for large dams.

g) Selection and placement of rip-rap. Rip-rap on earth and rockfill dams is usually constructed by pushing the larger rocks from the adjacent rockfill zone to the face of the embankment, and finishing the face of the embankment by carefully positioning rocks with an excavator. To satisfy rock grading requirements for rip-rap, it will be normal to specify the grading more closely than rockfill, e.g. requiring that at least 50% of the rock should be greater than a certain size. Thin rip-rap layers for earthfill dams may be placed progressively as the embankment is built, or on the completed face by dumping and spreading with a bulldozer.

14.2 SPECIFICATION OF EARTHFILL

It is common to specify the following for earthfill for dam construction:

a) The source of the earthfill.

b) The maximum particle size of hard clay, gravel or rock fragments in the earthfill. This will usually be specified as not greater than a size in the range 75 to 125 mm. The intention is to ensure that compaction is not affected by oversize material.

c) The particle size distribution. Some authorities specify this also: They require a minimum percentage passing the 0.075 mm sieve (i.e. silt and clay size). The figure used depends on the material available in the borrow area. Table 14.4 gives some examples. They may also require a minimum percentage passing the 4.75 mm sieve. This is to ensure the earthfill is not gap-graded, i.e. a mixture of clay and gravel, with no sand sized particles, because this may affect the design of Zone 2A filters.

d) An upper limit on fines. The authors' preference is to also specify an upper limit to fines for the earthfill, to ensure that the earthfill grading is compatible with the design of the Zone 2A filter. This may be done by specifying the grading envelope, as shown in Figure 7.14, or as a limiting percentage passing the 0.075 mm sieve, e.g. > 85% passing.

It is important to measure the particle size in the embankment after compaction to allow for breakdown of weak rock particles, and mixing of fine and coarse soils from the borrow area, rather than to test samples taken from the borrow area. During site investigations, the effect of

Table 14.3. Rockfill grading requirements.

Dam	Dam type	Zone	Grading Requirement			
			Max size (m)	Size largest particle (mm)	% other grading limits	
Cardinia Creek	E & R	3A	1.8	> 300	≤ 20% passing 25 mm	
Harding	E & R	3A	1.0	>150	≤ 30% passing 20 mm	≤ 10% passing 1.18 mm
Harris	E & R	3A	1.0	> 150	≤ 30% passing 20 mm	≤ 10% passing 1.18 mm
Thomson	E & R	3A	1.0	> 150	≤ 30% passing 19 mm	≤ 10% passing 1.18 mm
		3B	2.0	av. max size 1000	> 25% larger than 750 mm	≤ 25% passing 300 mm
Winneke	CFRF	3A	0.7	Not specified	≤ 10% passing 0.075 mm	
		3B random	0.4	Not specified	≤ 20% passing 0.075 mm	
Boondooma	CFRF	3A	0.6	> 75	≤ 15% passing 1.18 mm	≤ 5% passing 0.075 mm
–	–	3B	0.9	> 75	as above	as above

Note: 'Size largest particle' is to ensure some medium sized particles are present in the rockfill.

Table 14.4. Zone 1 earthfill material specification.

Authority/dam	Particle size			Atterberg limits		Adjacent foundation
	Max. size (mm)	% passing 0.075 mm	Other	Limit	Plasticity index	
MMBW Cardinia Creek	125	≥ 25	≥ 50% passing 4.75 mm	≤ 50%	≥ 10%	Finer, more plastic PI > 15%
PWD of WA Harding	100	Not specified	–	Not specified	Not specified	Finer, more plastic PI > 25%, WL > 40%
Water Authority of W.A. Harris	100	≥ 30% of that passing 4.75 mm, i.e. ≥ 15%	≥ 50% passing 4.75 mm	≤ 70%	≥ 10%	Finer, more plastic PI > 20%
MMBW Thomson	120	≥ 25	≥ 75% passing 4.75 mm	Not specified	≥ 5%	Finer, more plastic PI > 15%, WL ≤ 50%
Water Resourses Comm. of Q'Land Peter Faust	50	≤ 78	Full grading envelope specified	30 to 50%	> 10%	> 55% passing 0.075 mm
		≥ 30				WL 45-55 %, PI > 15%

such breakdowns can be assessed by carrying out particle size distributions on samples which have been subjected to compaction.

e) The Atterberg limits. Practice varies, most authorities specify a minimum plasticity index (see Table 14.4), and some also place an upper limit on liquid limit. This may reflect the presence of particularly high plasticity clays in the borrow area which may be hard to compact, but in general there should be no need to place an upper bound on the liquid limit. The authors' preference is to specify an allowable range of liquid limit and plasticity index by relating to the 'A' line on the plasticity chart. Figure 14.1 gives an example. The critical issue really is to specify limits which can be satisfied by the material you wish to use from the borrow area.

The Atterberg limit requirements should be seen only as a way of allowing rejection of unsuitable material if a dispute arises with the contractor. Generally, it should be possible to accept or reject material visually in the borrow area, at least after the job has been under way for some time and the contractor and supervisors are familiar with the materials.

f) Layer thickness, water content and density ratio. These are specified to ensure that the Zone 1 earthfill is compacted to a uniform low permeability material. Table 14.5 gives some examples from Australian dams. The following should be noted:

– Standard, not modified compaction should be used. This is to ensure moist compaction which leads to low permeability, flexible fills. Compaction at around modified optimum water content leads to high densities, but the soil structure is likely to be aggregated, leading to a higher permeability and more brittle fill

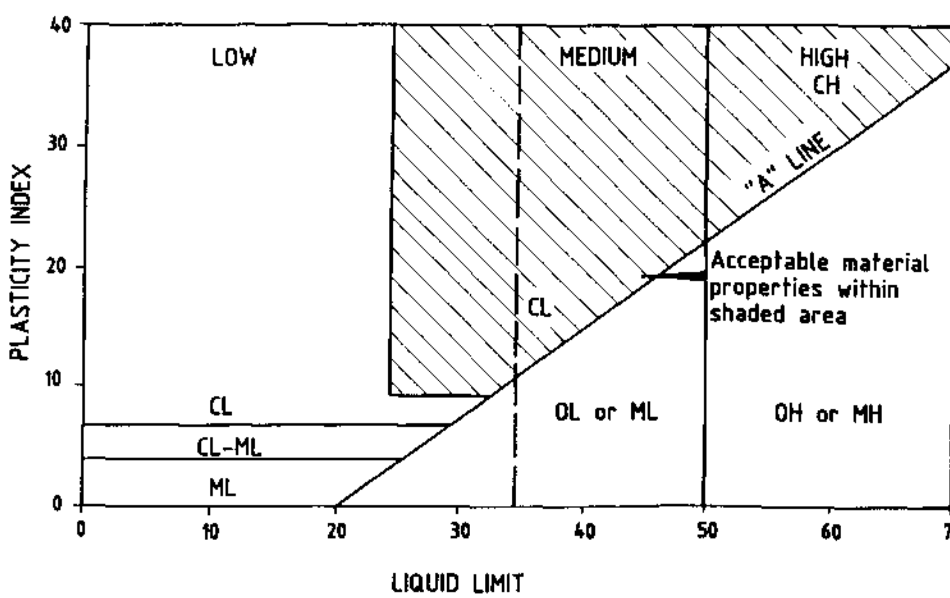


Figure 14.1. An example of specification of Atterberg limits for earthfill.

Table 14.5. Zone 1 earthfill – compaction specification.

Authority/dam	Layer thickness (mm) (1)	Water content (2)	Density ratio	Roller Type	Roller Weight	Roller Passes	Compaction requirements adjacent foundation
MMBW Cardinia Creek	150	OWC to OWC + 2%	≥ 98%	Tamping	6 tonnes/metre length	8 estimated	> OWC + 2% rubber tyred equipment
PWD of WA Harding	150	OWC – 1% to OWC + 1%	≥ 98%	Tamping or heavy rubber tyred	6 tonnes/metre length		> OWC + 2% rubber tyred equipment
Water Auth. WA Harris	150	OWC – 1% to OWC + 1%	≥ 98%	Tamping	Not specified	–	OWC + 1% to OWC + 3%
MMBW Thomson	150	OWC – 0.5% to OWC + 1.5%	≥ 98%	Tamping	6.2 tonnes/1 metre length	8 estimated	> OWC + 3% rubber tyred equipment density > 100% standard at the rolling water content
Water Resources Comm. Q'Land Peter Faust	150	OWC – 1% to OWC + 1%	10 consecutive tests: at least 9 ≥ 98%, all ≥ 97%	Tamping	Not specified	–	OWC to OWC + 2% rubber tyred equipment

(1) Thickness after compaction. (2) For standard compaction.

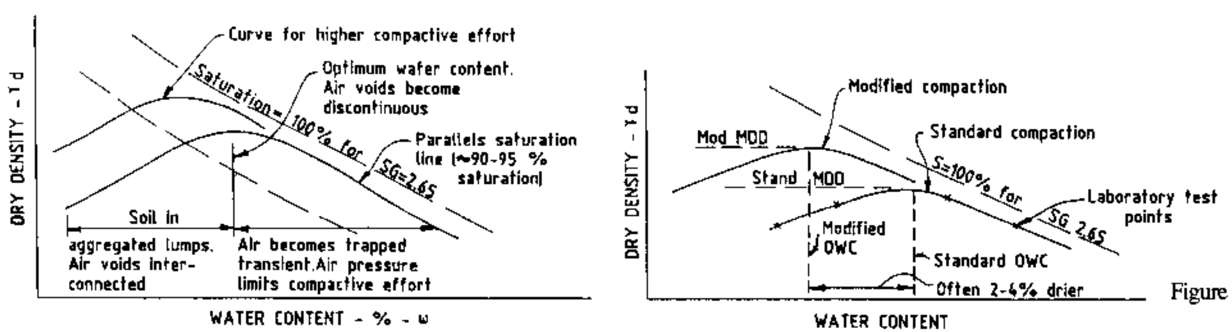


Figure 14.2. Compaction water content.

Table 14.6. Practical maximum layer thickness (m) after compaction for different types of rollers at different applications (Forssblad 1981).

Roller type static weight (drum module weights in brackets)	Embankment				Sub base	Base
	Rock fill(1)	Sand gravel	Silt	Clay		
Towed vibr. rollers						
6 tonne	0.75	*0.60	*0.45	0.25	*0.40	*0.30
10 tonne	*1.50	*1.00	*0.70	*0.35	*0.60	*0.40
15 tonne	*2.00	*1.50	*1.00	*0.50	*0.80	–
6 tonne padfoot	–	0.60	*0.45	*0.30	0.40	–
10 tonne padfoot	–	1.00	*0.70	*0.40	*0.60	–
Self-propelled vibr. rollers						
7(3) tonne	–	*0.40	*0.30	0.15	*0.30	*0.25
10(5) tonne	*1.50	*1.00	*0.70	*0.50	*0.40	
8(4) tonne padfoot	–	0.40	*0.30	*0.20	0.30	–
11(7) tonne padfoot	–	0.60	*0.40	*0.30	0.40	–
15(10) tonne padfoot	–	1.00	*0.70	*0.40	0.60	–
Vibr. tandem rollers						
2 tonne	–	0.30	0.20	0.10	0.20	*0.15
7 tonne	–	*0.40	0.30	0.15	*0.30	*0.25
10 tonne	–	*0.50	*0.35	0.20	*0.40	*0.30
13 tonne	–	*0.60	*0.45	0.25	*0.45	*0.35
18 tonne padfoot	–	0.90	*0.70	*0.40	0.60	–

(1) For rock fill only rollers especially designed for this purpose.
Note: Most suitable applications marked *.

– The requirement for density ratio ≥ 98% is reasonable and compatible with the water content ranges shown. There is no advantage in specifying a higher density ratio, and it may be detrimental in that the contractor will be forced to compact dry of optimum water content. For smaller dams, and dams to be constructed in wet climates, it would not be unreasonable to relax the compaction requirement to as low as 95% density ratio provided that compaction is carried out above optimum water content. However, compaction to only 95% density ratio at say optimum -3% would lead to a permeable soil structure and would not normally be acceptable. Figure 14.2 shows some of these effects

– The 150 mm layer thickness used for the dams in Table 14.5 is probably unnecessarily thin for modem heavy rolling equipment. Table 14.6 from Forssblad (1981) shows practical maximum layer thicknesses for a range of roller types and weights, and soil types indicating that 6 tonne to 10 tonne pad foot rollers can compact layers up to 0.3 and 0.4 m thick respectively. While these figures may represent an upper practical limit, it does show that 150 mm layers is unnecessarily restrictive. It could be argued that as a performance specification has been adopted for earthfill, i.e. water content and density ratio, there is no need for the 'method or procedure specification' limitation on layer thickness. The authors' own preference is to specify layer thickness but to use more realistic figures, e.g. 0.2 or 0.25 m for a 6 tonne tamping roller.

g) Roller type and weight. It is normal to specify the roller type and weight. For earthfill it is most common to require a tamping foot (or 'sheep's foot' or 'pad foot') type roller because:

– The tamping foot action breaks up pieces of cemented soil or weathered rock in the fill.

– The rough surface of the layer left by a tamping foot layer allows better bond between layers.

– There is less tendency for formation of large shear surfaces beneath the roller when a tamping foot roller is used. This tendency is a particular problem in high plasticity, fine grained soils placed wet of optimum.

– The compaction water content is less critical than for rubber tyred rollers, i.e. the shape of the dry density-water content curve is broader for tamping feet type rollers.

The weight is best specified as shown in Table 14.5, i.e. as a weight per metre length of roller, as this better defines the contact pressure than total weight. Some authorities also specify the geometry and arrangement of the 'feet' on the roller. This would seem to be unnecessarily restrictive for the majority of dam construction.

Sherard et al. (1963) report rolling trials carried out at the US Army Experiment Station, which showed that the foot size did affect the maximum dry density and optimum water content (as did the tyre pressure for rubber tyred rollers). Again, it could be argued that provided the performance specification is defined, there is no need to also specify roller weight and type.

There are occasions where Zone 1 earthfill is better compacted with different roller types, e.g:

– Where water contents are naturally high and compaction is required at well above optimum water content. In these cases a rubber tyred roller may be preferable as it will be less likely to become clogged with the wet, sticky clay. Gibbs (1982) used low ground pressure swamp dozers to compact very high water content (80 to 120%) halloysitic clays in the Monasavu Dam in Fiji. No rollers were used, as the rolling action released water from the soil structure causing equipment to bog. Such techniques lead to compressible cores with high pore pressure, but they perform the primary function of low permeability. Sherard et al. (1963) describe similar techniques used for wet, silty, sandy gravel glacial soils in Sweden. Penman (1983) also describes some recent dams where materials with high water content have been placed.

– Where the fill is a weathered rock which breaks down to a clayey silty sand/silty sand, steel drum rollers, compacting in thin layers (150 mm) may give better breakdown of the rock and compact it more readily. Best performance in these cases may be obtained without vibration.

h) Fill adjacent to foundation. It is normal to specify that the earthfill which is to be placed adjacent (within say 0.6 m) of the dam foundation, is to be composed of finer more plastic soil available from the borrow area, and is to be compacted at a higher water content (e.g. OWC + 2% or OWC + 3%) with rubber tyred construction equipment or rollers. This is to facilitate squeezing the soil into the irregularities in the foundation. Generally there is no 'performance' specification (i.e. density ratio), because the layers are thin and testing is impracticable. On Thomson Dam, Snowy Mountains Engineering Corporation (SMEC), designing for the Melbourne and Metropolitan Board of Works (MMBW), required that the contact zone be compacted to a density 100% of that which would be achieved by compacting in the laboratory with standard compactive effort, at the field water content. This is a reasonable approach which allows for the high compaction water content, but should not be specified except for such large dams as Thomson (160 m high).

i) Water content adjustment. Most specifications require that water content adjustment be carried out in the borrow area, with only minor adjustment allowed on the embankment. This means that dry (or wet) soil in the borrow area is brought to a uniform water content by irrigating, harrowing, watering (or drying), and reworking before transportation to the embankment. It is impractical to do this on the embankment. Soils which are particularly dry or wet of the required water content may have to be conditioned for some days before use in the embankment.

Sherard et al. (1963) give details of procedures which may be necessary to raise the water content of dry soils in the borrow area. They indicate that:

– Irrigation may be achieved by ponding of water or spray irrigation. Ponding is only suited to flat areas, and can result in large evaporation losses:

– The water content seldom becomes too high – most soils only take in water up to about the optimum.

– Ripping to 0.6 to 0.9 m will assist in allowing water to penetrate. Contour ploughing can assist in hilly borrow areas.

– Water conditioning up to 1.5 to 4.5 m depth has been successfully achieved.

Earthfill which dries out, or gets wet on the embankment must be tyned, the moisture adjusted, and thoroughly reworked with a grader, before recompacion. If the soil is judged too wet or dry to be adjusted on the embankment, it should be removed from the embankment.

The tendency for wetting of earthfill on the embankment during rain, is usually reduced by requiring the surface to be 'sealed' with a smooth drum roller if rain is about to fall, and the surface contoured to allow runoff of surface water. The sealed surface must be tyned prior to placing the next layer.

j) Compaction of edges of fill. Under normal operations, the outer 1 to 1.5 m (measured horizontally) of an earthfill embankment will not be adequately compacted by rollers. It is necessary to specify either that the embankment is constructed oversize, and trimmed back to the required lines by removing this poorly compacted soil, or to require rolling of the surface up and down the slope.

If left in place, the poorly compacted soil will often soften and lead to surficial sliding, particularly on steeper slopes. While this will not in itself lead to failure of the embankment, subsequent erosion or sliding may cause problems.

14.3 QUALITY CONTROL

14.3.1 *General*

The question of quality control for earth, earth and rockfill, and rockfill dams has recently been addressed by ICOLD and reported in Bulletin 56 (ICOLD, 1986b). The following discussion summarises the concepts in that bulletin, supplemented by some opinions of the authors' and data from Australian practice.

14.3.2 *'Methods,' and 'Performance' criteria*

As has been discussed in the earlier part of this chapter, there are two principal types of technical specifications:

a) Method or procedure specifications which describe how the construction is to be carried out, in order to achieve the desired end product. The specifications stipulate to the contractor the materials to be used, the equipment and construction procedures, e.g. specification for earthfill includes:

– the source of materials

– the water content
– layer thickness

– roller type and weight
– number of passes.

This places the onus on the owner/engineer to have established procedures which will yield a satisfactory product.

b) Performance or end product specifications which describe the end result to be achieved by the contractor. It is the contractor's responsibility to select materials, equipment and methods to obtain the specified end product, e.g. specification for earthfill includes:

– particle size gradation
– Atterberg limits
– layer thickness
– water content
– density ratio.

Many dam specifications are a mixture of these two alternatives, and as has been discussed above, this is often unnecessary and can lead to inefficient construction procedures (e.g. requiring too thin a layer for compacting earthfill), unnecessary costly testing (as in requiring particle size and density of rockfill), and disputation between constructor and owner engineers when the 'methods' part of the specification fails to produce the required performance criteria.

In most projects, methods specifications become the routine quality control technique, even if a performance specification has been used for the contract documents. This is the only practical way of allowing work to proceed without unnecessary delay.

On most projects the owner/engineer will be responsible for the quality control, i.e. the inspection, testing, assessment whether the required quality standard has been achieved, and the authority to require additional work to achieve the standard or to reject the non conforming work. The owner/engineer will also be responsible for quality assurance, i.e. that the quality control standards are valid for the dam, that the specified tests are being implemented and correctly performed, that the quality control plan is working, and that records and reports are verified and maintained.

On some projects the quality control aspect may be incorporated into the contract and be the contractor's responsibility. This can be quite satisfactory provided that the contract documents clearly define the level of inspection and testing required, the standards to be adopted, and the inspection and testing group are given authority by the contractor's management to enforce the standards. Often, in these cases, the testing will be subcontracted to a specialist geotechnical consultant. This assists in maintaining the autonomy of the inspection and testing group from the main contracting staff.

14.3.3 *Quality control*

a) Inspection. Inspection must always form a critical part of any quality control plan. The field and laboratory testing program should be seen as first establishing the methods required to achieve the required quality, then to ensure that the quality is being maintained, and as a definitive, quantifiable means of rejecting substandard work. It is clearly impractical to test the whole of the completed product so one must rely on visual inspection to maintain overall quality.

It is important that the inspectors are properly trained and briefed on the implications of substandard work. It is also important to recognise that inspectors will often be needed in the borrow areas, as well as on the embankment, so that unsuitable material can be rejected before it reaches the embankment.

b) Testing. Most testing which is carried out is for quality control, i.e. to ensure that the requirements of the specification are being met. The selection of areas or materials for testing may be done in two ways:

– Selecting those areas which are judged by the supervisor/inspector to be least likely to meet the specification. This can assist in reducing the quantity of testing and, if the testing shows acceptable performance, should ensure the overall adequacy of the construction work. It does, however, require independent and experienced supervisors.

– Selecting test areas at random, at the minimum recommended frequency. This will yield test results which better reflect the overall condition of the construction work, as the biased sampling of the first alternative is removed.

The latter method is better suited to establishing statistical limits to the testing, allowing recognition of the fact that there is a statistical sampling error, and that within a large mass of earth and rockfill, a small proportion of the material failing to meet the basic specification criteria will not affect overall performance.

Which of these two methods is adopted is also related to the specification limits used, and whether the supervisors are given any latitude in accepting material which falls below the specification.

For example, if earthfill is specified to have a density ratio in excess of 98%, with a water content optimum -1% to optimum plus 1%, it would be reasonable in most cases to accept an area which has tested at 97% density ratio, OWC + 2%, as this will still yield a low permeability fill. However, one would almost certainly reject an area which has tested at 95% density ratio, at OWC -3%, as this will give a more permeable fill.

Similarly, if a relatively low compaction standard is set – say 95% density ratio, no 'failures' should be accepted.

The authors' opinion is that it is desirable to establish such guidelines in the design stage of a project, and to detail them in the specification either in a descriptive form or statistical form. Alternatively a rigid specification limit may be set, but the site supervision staff should be advised as to the degree of flexibility which can be used in applying the specification.

The question of how many tests should be carried out on earth and rockfill is virtually impossible to answer, as it is interrelated to the specification standards, the competence of the contractor and the inspectors, and the site conditions (e.g. variability of materials, climate).

ICOLD (1986) gives some useful examples of minimum frequency of testing required for some large fill dams in USA, Austria, India, Canada and Italy. Tables 14.7, 14.8 and 14.9 summarize this information. Also shown is the testing specified for the random fill zone in Winneke Dam by the Melbourne and Metropolitan Board of Works, and for the earthfill in the Ranger Mine Tailings dam (a zoned earth and rockfill dam).

c) Reporting. It is important that complete records should be kept of all construction operations. These are invaluable in the event that repairs or modifications are required, if the embankment is to be raised in the future, and for surveillance during the life of the dam. Records are also important in respect to contractual and insurance claims. The reporting should include:

– plans and specifications, including amendments and work as constructed;
– final construction report written by the engineer;
– monthly progress reports, and reports on technical meetings;
– reports from dam review panel;

Table 14.7. Minimum required frequency of construction testing – Zone 1 earthfill.

Dam	Country	Fill volume	Particle size	Atterberg limits	Water content	Density ratio	Permeability	Shear strength
Oroville	USA	8,649,000 yd^3	4,000 yd^3	4,000 yd^3	4,000 yd^3	4,000 yd^3	150,000 yd^3	–
Culmback	USA	27,000 yd^3	1,800 yd^3	–	500 yd^3	1,000 yd^3	6,750 yd^3	13,500 yd^3
Beas	India	11,500,000 m^3	56,000 m^3	56,000 m^3	1,150 m^3	1,150 m^3		56,000 m^3
QA-8, La Grande	Canada	1,900,000 m^3	4,000 m^3	If reqd.	1,000 m^3	4,000 m^3	2 tests/season	800,000 m^3
Brandy Ranch	USA	1,270,000 yd^3 (1)	–	4,000 yd^3	4,000 yd^3	4,000 yd^3	–	–
Bloomington	USA	1,750,000 yd^3	5,000 yd^3	5,000 yd^3	5,000 yd^3	5,000 yd^3		15 tests
Winneke	Australia	– (2)	15,000 m^3 or 2 per week	15,000 m^3 or 2 per week	5,000 m^3 or 1 per day	5,000 m^3 or 1 per day	–	–
Ranger Mine Tailings Dam (3)	Australia	135,000 m^3	3,000 m^3	3,000 m^3	1 per shift	500 to 1000 m^3	Some	Some

Notes: (1) Earthfill and random fill; (2) Random fill; (3) Embankment is zoned earth and rockfill.

Table 14.8. Minimum required frequency of construction testing Zone 3A and 3B type rockfill.

Dam	Country	Dam type	Rockfill volume	Particle size	Compaction test	Density in place	Permeability	Shear strength
Oroville	USA	E & R	60,300,000 yd^3	each 24 hrs	100,000 yd^3	100,000 yd^3		
Culmback	USA	E & R	130,000 yd^3 (1)	5,200 yd^3	7,200 yd^3	7,200 yd^3	2 tests	2 tests
Finstertal	Austria	Asphalt core	2,790,000 m^3 –	100,000 –	Nil zone 3B 100,000 m^3 zone 3A	2 tests per season	3 tests zone 3B 100,000 m^3 zone 3A	2 tests/season 1 test/season
Zirmsee	Austria	Asphalt face	550,000 m^3	40,000 m^3	–	2 tests	–	–
Beas	India	E & R	22,400,000 m^3	–	–	15,000 m^3	A number tests	A few tests
QA-8, La Grande	Canada	E & R	5,600,000 m^3	4,000 m^3	10,000 m^3	5,000 m^3	–	–
Bloomington	USA	E & R	3,010,000 yd^3	100,000 yd^3	–	100,000 yd^3	–	–
Anapo upper and lower	Italy	Asphalt face	700,000 m^3	15,000 m^3	15,000 m^3	45,000 m^3	30,000 m^3 (1.5 m. dia.) 100,000 m^3 (0.6 m. dia.)	–
Winneke	Australia	CFRF	–	50,000 m^3 or 1 per week 25,000 m^3 or 2 per week or random fill	– –	50,000 m^3 or 1 per week 25,000 m^3 or 2 per week or random fill	N/A	N/A

Notes: (1) Gravel fill. (2) – indicates data not available.

Table 14.9. Minimum required frequency of construction testing Zone 2A and 2B filters.

Dam	Country	Filter volume	Particle size	Relative density	Permeability	Shear strength
Oroville	USA	9,500,000 yd^3	each 24 hrs	4,000 yd^3	150,000 yd^3	–
Culmback	USA	36,000 yd^3	1,400 yd^3	1,400 yd^3	2 tests	2 tests
Beas	India	1,670,000 m^3	–	6,000 m^3	Occasionally	–
QA-8, La Grande	Canada	1,300,000 m^3	4,000 m^3	4,000 m^3	–	–
Brandy Ranch	USA	22,000 yd^3	1,500 yd^3	1,500 yd^3	–	–
Bloomington	USA	444,000 yd^3	4,000 yd^3	5,000 yd^3	–	–
Winneke	Australia	Filter blanket	1,000 m^3 or each layer	2,000 m^3 or each alternate layer	N/A	N/A
Ranger Mine Tailings Dam	Australia	297,000 m^3 (1)	1,000 m^3 to 2,000 m^3	Method specification	Some	

(1) Filter transition, combines function of Zones 2A & 2B.

– laboratory test reports, including clear definition of location and level of samples tested, and differentiating between original tests and retests after failures;

– daily reports by all supervisory personnel and inspectors in the form of diaries. These should concern adequacy of progress and comments on decisions.

The authors have been involved in a major dam foundation failure where these daily reports proved invaluable.

14.3.4 *Influence of non technical factors on the quality of embankment dams*

In the Casagrande volume (Hirschfeld & Poulos 1972), Professor Ralph B.Peck highlighted some of the 'facts of life' relating to the influence of non technical factors on construction of embankment dams. Professor Peck points out that many shortcomings in dam engineering relate to the attitudes and actions of the owner, designer, contractor and technical consultants and these can outweigh the real technical issues. His comments are still relevant today, and are recommended reading for all who are involved in dam engineering.

14.4 TESTING OF ROCKFILL

14.4.1 *Particle size, density and permeability*

The testing of rockfill to determine the particle size, compacted density and permeability is complicated by the large size of the rock particles.

Bertram (1972) describes field density tests in rockfill with a maximum size of 0.45 m, using a 1.8 m diameter density ring to define the density in place hole. The hole was dug to the full depth of the layer. ICOLD (1986) give details of testing for Oroville Dam, where a 1.8 m diameter ring was used for rockfill up to 0.6 m size. Hence, a ring size 3 to 4 times the maximum particle size has been used to get representative samples. For equivalent samples of rock up to 1 m maximum size, a ring and hole 3 m diameter would be needed, giving about 15 tonnes of rockfill to be excavated for a 1 m thick layer. This highlights the magnitude of the problem, and is the reason why it is impracticable to carry out density and particle size distribution tests in rockfill for smaller dams.

Penman (1983) and Forssblad (1981) describe a compaction meter, which uses an accelerograph to monitor the response of a vibratory roller and assists in determining when the required degree of compaction has been achieved.

Permeability of rockfill can be determined by ring infiltration tests *in situ*. If the rockfill is permeable the quantities of water involved would be huge and the results of doubtful value. For low permeability rockfill, tests through say 1.0 to 1.8 m diameter rings, depending on particle size, could be carried out. In most cases a subjective assessment, by observing whether water will pond on the surface or in a hole dug through a layer, will be adequate to ascertain if the fill is 'free draining' or not. Bertram (1972) and ICOLD (1986) give references for field testing of rockfill.

14.4.2 *Field rolling trials*

Field rolling trials are often carried out for larger dams:

a) before construction, to ascertain the degree of breakdown of rock under rollers, the

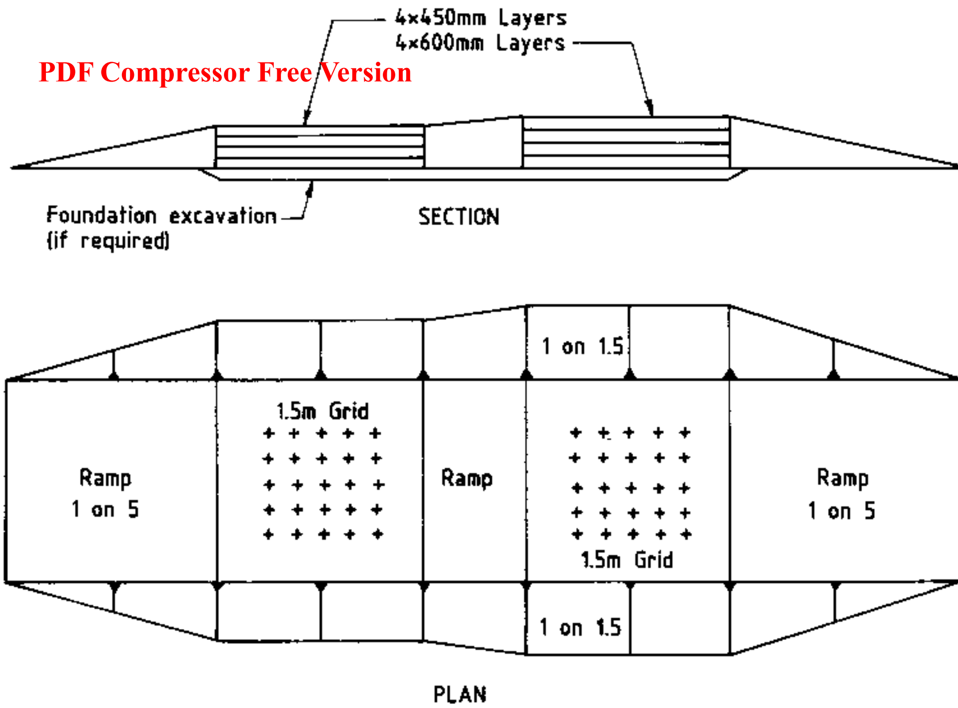


Figure 14.3. Layout of rockfill roller trial (Bertram 1972). Reprinted by permission of John Wiley & Sons Inc.

resulting density and particle size, distribution, permeability and modulus of compressibility;

b) during construction, to determine the number of passes of the roller required to achieve the required density of compaction.

Figure 14.3 from Bertram (1972) details a layout for roller trials on rockfill.

Bertram makes the following points:

1. An area sufficient to give 25-30 measuring points is necessary to overcome non uniformity in the rockfill.

2. The grids can be set out as shown (1.5 m) or 1.2 × 1.3 m or 1.5 × 2.1 m.

3. Measurements should not be taken closer than 3 m from the edge.

4. Several layers (4 to 5 minimum) are required.

For construction, the specification usually requires that trials be carried out on the embankment, requiring an area of about 200 m^2.

In the trials, the settlement of the surface of the rockfill is measured after each pass of the roller, and the results plotted as average settlement (or % settlement) vs number of passes. Figures 14.4 and 14.5 show the results of roller trials at Murchison Dam, and at Boondooma Dam.

The Boondooma Dam trials show the effect of watering the rockfill.

It can be seen that the additional compaction achieved after, say, 4 or 6 passes is relatively small. Since this additional compaction is often being achieved largely by breakdown of the upper part of the layer, it is common to limit the number of passes to 4 or 6, seldom more than 8.

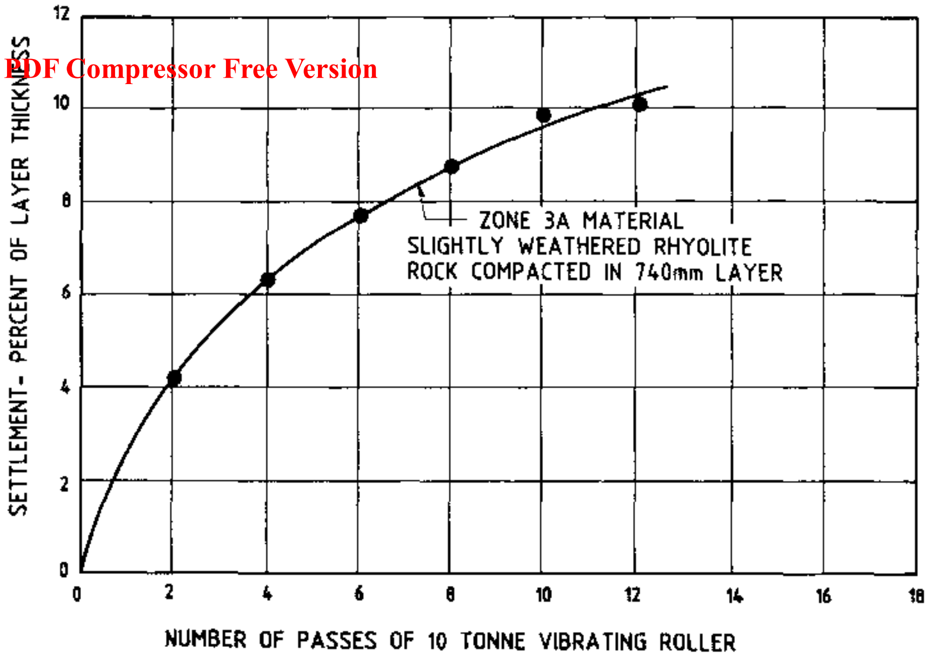


Figure 14.4. Murchison Dam rockfill roller trials (Fitzpatrick et al. 1985).

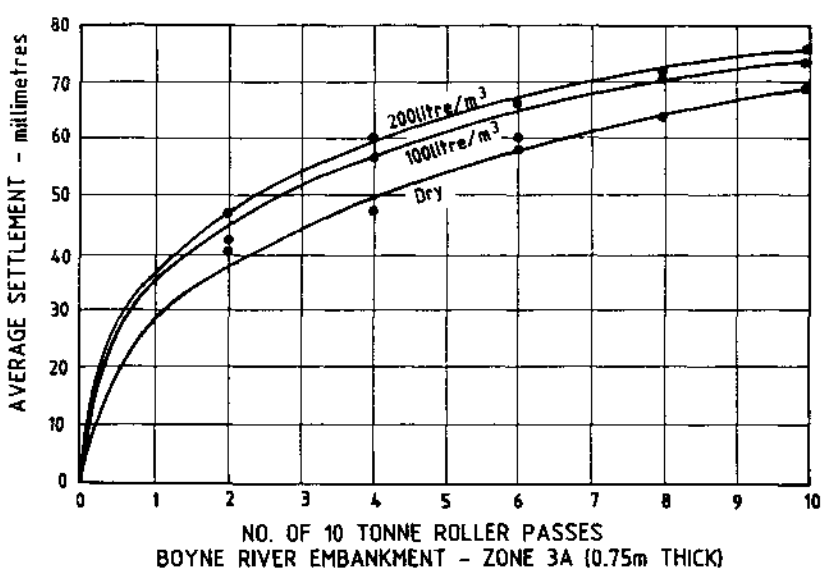


Figure 14.5. Boondooma Dam rockfill roller trials (Rogers 1985).

14.5 TESTING OF EARTHFILL

14.5.1 *Compaction-test methods*

The degree of compaction of cohesive earthfill is determined by the density ratio, where

$$\text{density ratio} = \frac{\text{dry density in place}}{\text{maximum dry density}}$$

For earthfill in dams, the maximum dry density should be obtained using the standard compaction method (also known as standard Proctor) using the standard applied in the country, e.g.

AS1289 E1.1 – Australia
ASTM D698-78 – USA
BS1377 4.1 Test 12 – United Kingdom.

The *in situ* density should be determined using the sand replacement, rubber balloon or core cutter method

AS1289 E3.1, E3.2 and E3.3
ASTM D1556-32, D2167-66 and D2937-71
BS1377 4.1 Tests 5, 15.

For cohesionless soil (sand, silt, sand/gravel) the degree of compaction is determined by density index, (or relative density), where

$$\text{density index} = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\%$$

where e = voids ratio in place
e_{max} = voids ratio in loosest state
e_{min} = voids ratio in most compact state
also

$$\text{density index} = \frac{\gamma_{dmax}(\gamma_d - \gamma_{dmin})}{\gamma_d(\gamma_{dmax} - \gamma_{dmin})} \times 100\%$$

where γ_d = dry density in place
γ_{dmax} = dry density in most compact state
γ_{dmin} = dry density in loosest state.

The maximum and minimum dry densities are determined in the laboratory using the standard applying to the country, e.g.

AS1289 E5.1
ASTM D2049-69
BS1377 4.3 Test 14.

These methods all require the water content to be determined using an oven, and this usually takes about 24 hours at 110°C which may be unacceptably long for the construction situation. For this reason it is common to specify that routine quality control will be carried out using the Hilf method. This method is described in detail in USBR (1985) and is a standard test (AS1289 E7.1).

The method allows approximate determination of the density ratio within 1 hour of the density in place test, and accurate determination the next day when the water content is confirmed. Some specifications allow for disputes to be settled based on the measured water content.

Other methods which can be used to allow a more rapid determination of water content, and, hence, density ratio, include drying the soil by:

– microwave oven
– methylated spirits
– high temperature oven
– heating the soil on a hot plate with gas burners.

These methods are more approximate, and require calibration between the method and water contents determined in the standard oven technique, to allow for adsorbed water being driven off by high temperature drying.

An alternative method of routine compaction control is the use of nuclear methods for determination of density and water content.

In this method gamma rays are used to determine the density, and neutrons are used to determine the water content. This may be done by direct transmission, backscatter or less commonly the air gap methods. The most common method is backscatter, because of the difficulty of penetrating the source into the compacted fill.

The method requires calibration against compacted materials of known density and water content from the site. Provided this is done, and the earthfill is not too variable or had too high a gravel content, the method can be adequate for routine control.

14.5.2 *Compaction control – Some common problems*

Some common problems which arise in compaction control, and which lead to disputation between contractor and engineer include:

a) Specifying too high a compaction standard, e.g. 98 or even 100% density ratio, modified compaction, for clay soils or 100% density index for granular soils. This is virtually unobtainable, even with very heavy rolling equipment and, as discussed above, is undesirable for dam construction because it can only be achieved by compacting the clay fill dry, resulting in a brittle, permeable fill, or by overcompacting granular filter materials giving excessive breakdown.

b) Specifying unnecessarily restrictive water content range. Specification limits must be realistic to match the available materials. If the soil in the borrow area is, say, 4% wet of optimum in the borrow area, and the climate is wet, it is pointless requiring 98% density ratio at OWC ± 1%. The specification would be more realistically 95% density ratio at optimum to optimum + 4%.

c) Carrying out insufficient laboratory compaction tests. The density ratio is obtained by comparing the density in place with the maximum dry density obtained in the laboratory. Ideally the soil for this laboratory compaction is sampled beside the density in place test. However, it is common practice to reduce the work involved, by doing one laboratory compaction for every 2 to 4 or more density in place tests, assuming the soils are uniform. On some projects very few laboratory tests are done, as the borrow area is assumed to be uniform. This is almost invariably not true and, as a small change in the maximum dry density can make the difference between acceptance and failure, disputes often arise when such short cuts are used.

d) Breakdown of materials during compaction. Soils which contain pieces of weathered rock, or gravels which break down under compaction, will often give a higher laboratory maximum dry density if the compaction test is carried out on soil dug from adjacent the density in place test hole, i.e. on soil already compacted and broken down by the roller, than if the test is carried out on material sampled direct from the borrow area. The result of this is that the density ratio calculated is lower when the recompacted soil maximum dry density is used, often resulting in rejection of the fill. Since the rollers are dealing with the soil from the borrow area, not recompacted soil, this is unreasonable. There are two solutions for this problem, either lower the density ratio standard and use the recompacted maximum dry density, or ensure that laboratory compactions are done on representative uncompacted material. The latter may be difficult because of material variability.

e) Property change on drying. Some soils change properties when dried in an oven or under lights. Halloysite clays are particularly prone to this, but most clays are affected. It is desirable not to completely dry the soil used for the laboratory compaction test, only to dry it to the water content needed for testing.

f) Vibration from nearby construction equipment may affect the density obtained by sand replacement methods. This can be overcome by using the water balloon method, or testing when equipment is not operating nearby.

g) Inadequate curing of samples. Soils, particularly higher plasticity clays, need time for water added for laboratory compaction tests to evenly distribute throughout the sample. This is the reason why most standards require 12 to 24 hours 'curing' of the soil before compaction. In a construction situation this may be regarded as impracticable, and not adhered to. As a result, compaction results may be inconsistent and subject to error.

h) Specification of standard soil tests for 'gravelly' materials. The standard tests can be corrected for the presence of gravel particles up to a reasonable limit. However, if the soil to be tested is largely gravel size, the potential errors are too great, and larger size compaction moulds must be used, or a methods type specification adopted. Inexperienced persons may even specify a density ratio rather than density index (relative density) for granular soils. One need only to observe the loosening effect of a compaction hammer on sand in a compaction mould to appreciate that this cannot work.

14.5.3 *Compaction control – Some other methods*

Some other approaches have been developed for routine compaction control of clay soils, particularly for fine grained clay soils which are compacted wet of optimum water content. These include:

– Specifying water content, and a density ratio based on wet density. The authors have used this approach for a dam constructed in Papua New Guinea, using halloysitic clay where previous experience had shown that it was very difficult to define maximum dry density and water content, i.e. the laboratory testing gave very variable results. The specification was written to require a water content between 44 and 50% (roughly OWC ± 3% for the mean OWC), and the required compaction dry density was 98% of the dry density achieved in the laboratory at the field water content. This wide range of water content was only practicable for the soil being used and would not normally be acceptable

– Specifying undrained shear strength of the compacted earthfill. Knight (1990) describes the use of this technique, which is common in dams constructed in the United Kingdom. Undrained strengths used range from 40 to 110 kPa, with tests being carried out using triaxial tests on 100 mm diameter driven tube samples, or on remoulded samples. Lower bound and mean values are specified. Knight (1990) indicates that advantages of the method are that measured strengths reflect design intention, and testing 'is speedy.' The authors' view is that the technique is useful where soils are to be compacted significantly wet of optimum, but that otherwise it is preferable to adopt a density ratio and water content specification. For most dams, the undrained shear strength of the core is not a critical issue, because stability is controlled largely by rockfill zones.

CHAPTER 15

Design of dams to withstand earthquakes

15.1 EFFECT OF EARTHQUAKE ON EMBANKMENT DAMS

Earthquakes impose additional loads on to embankment dams over those experienced under static conditions. The earthquake loading is of short duration, cyclic and involves motion in the horizontal and vertical directions. Earthquakes can affect embankment dams by causing any of the following:

– settlement and cracking of the embankment, particularly near the crest of the dam;

– reduction of freeboard due to settlement, which may, in the worst case, result in overtopping of the dam;

– instability of the upstream and downstream slopes of the dam;

– differential movement between the embankment, abutments and spillway structures, increasing the likelihood of leakage and piping failure;

– liquefaction or loss of shear strength in the embankment and its foundations due to increase in pore pressures induced by the earthquake;

– differential movements on faults passing through the dam foundation;

– overtopping of the dam in the event of large tectonic movement in the reservoir basin, by seiches induced upstream;

– overtopping of the dam by waves due to earthquake induced landslides into the reservoir from the valley sides;

– damage to outlet works passing through the embankment leading to leakage and potential erosion of the embankment.

The potential for such problems depend on:

– the seismicity of the area in which the dam is sited,

– local foundation and topographic conditions,

– the type of dam,

– the size of the embankment.

The amount of site investigation, design, and additional construction measures (over those needed for static conditions) will depend on these factors, and the hazard rating of the dam. It is important to recognise at the outset that most dams will not be significantly affected by earthquake and do not warrant extensive investigation and design measures. ICOLD (1986c) quote Seed (1979): '(a) Hydraulic fill dams have been found to be vulnerable to failures under unfavourable conditions and one of the particularly unfavourable conditions would be expected to be the shaking produced by strong earthquakes. However, many hydraulic fill dams have performed well for many years and when they are built with reasonable slopes on good foundations they can apparently survive moderately strong shaking – with accelerations of about 0.2 g from magnitude 6.5 earthquakes with no harmful effects. (b) Virtually any well-built dam on a firm foundation can withstand moderate earthquake shaking, say with peak accelerations of about 0.2 g, with no detrimental effects. (c) Dams constructed of clay soils on clay or rock foundations have withstood extremely strong shaking ranging from 0.35 to 0.8 g from a magnitude 8 earthquake with no apparent damage. (d) Two rockfill dams have withstood moderately strong shaking with no significant damage and if the rockfill is kept dry by means of a concrete facing, such dams should be able to withstand extremely strong shaking with only small deformations. (e) Dams which have suffered complete failure or slope failures as a result of earthquake shaking seem to have been constructed primarily with saturated sand shells or on saturated sand foundations. (f) Since there is ample field evidence that well-built dams can withstand moderate shaking with peak accelerations up to at least 0.2 g, with no harmful effects, we should not waste our time and money analysing this type of problem – rather we should concentrate our efforts on those dams likely to present problems either because of strong shaking involving accelerations well in excess of 0.2 g or because they incorporate large bodies of cohesionless materials (usually sands) which, if saturated, may lose most of their strength during earthquake shaking and thereby lead to undesirable movements. (g) For dams constructed of saturated cohesionless soils and subjected to strong shaking, a primary cause of damage or failure is the buildup of pore water pressures in the embankment and the possible loss of strength which may accrue as a result of these pore pressures. It is not possible to predict this type of failure by pseudostatic analyses, and other types of analysis techniques are required to provide a more reliable basis for evaluating field performance.'

The aim in this chapter is to present an overview of the topic, rather than to give detailed methods, because these are far too complex and extensive to be covered here. The authors have drawn largely from several useful review papers including Seed (1979), ICOLD (1983), ICOLD (1986c), USNRC (1985), Martin (1988) and Finn (1990).

15.2 ASSESSMENT OF DESIGN EARTHQUAKE

15.2.1 *Measurement of earthquake strength*

Earthquakes are measured in terms which are mentioned in the following sections.

15.2.1.2 *Magnitude*

This is a quantitative value obtained from seismographs, and reflects the total energy radiating from the focus of an earthquake.

There are two measures

$$\text{Local magnitude} \quad M_L = \log_{10} A$$

where A = maximum seismic wave amplitude (in thousandths of a millimetre) recorded from a standard seismograph at a distance of 100 km from the earthquake epicentre; or

$$M_L = \log_{10} A - F(\Delta) + k$$

where $F(\Delta)$ = distance correction

k = scaling constant.

This allows calculation of M_L at different distances from the earthquake.

Body wave magnitude $M_b = \log V + 2.3 \log \Delta$

where V = maximum ground velocity in microns/sec recorded by the seismograph

Δ = distance from epicentre.

Earthquakes with a magnitude of less than 3 or 4 will usually not cause any felt effect, and earthquakes with a magnitude less than about 5 will usually not cause any damage. The maximum recorded magnitude is ≈ 8.9. The scale is not linear. Each step in the magnitude scale represents a thirtyfold increase in energy released by the earthquake e.g. a magnitude 5 earthquake represents 900 times the energy of a magnitude 3, and a magnitude 8 represents 10^6 times the energy of a magnitude 4.

The magnitude, epicentral location, and focal depth for an earthquake are determined from seismographs. Three or more instruments are needed to allow triangulation.

Worldwide records are available from the US Geological Survey, Colorado, and the International Seismological Centre, Newbury, England. In the Australian region, records are available from the Bureau of Mineral Resources, Canberra. Good data is available for events of the last 50 years, but whether small seismic events are recorded will depend on the proximity of instruments at the time, particularly with older records.

15.2.1.2 *Intensity*

Earthquake intensity is a qualitative value based on the response of people and objects to the earthquake. The intensity depends on distance from the earthquake, ground conditions and topography, so there will be a range of intensity values for any earthquake. The most commonly used scale is the modified Mercalli Scale which is reproduced in Table 15.1.

15.2.1.3 *Acceleration*

For the design of dams, the horizontal and vertical acceleration induced by the earthquake at the base of the dam is usually required. Information is best obtained from accelerograph measurements at the dam site, but in many cases will be obtained from records of sites with similar geological conditions. A typical accelerograph record is shown in Figure 15.1.

Acceleration is often quoted as a multiple of the acceleration due to gravity, e.g. 0.1 g. The response of the dam is dependent on the amplitude of the ground accelerations, the duration of ground motion, and the frequency of the cycling.

There are no unique relationships between magnitude, intensity and acceleration as these depend on the energy released by the earthquake, the source mechanism of the earthquake, the geological conditions between the earthquake and the site, the distance between the earthquake and the site and geological conditions and topography at the site. A rough approximation for hard ground sites not affected by local topography can be obtained from Figure 15.2. In this figure, acceleration and intensity are at the earthquake epicentre.

15.2.2 *Attenuation and amplification*

15.2.2.1 *Attenuation*

The effect of an earthquake is attenuated with distance from the epicentre. There are several equations available based on recorded events, e.g. Esteva & Rosenblueth (1969) suggest that ground acceleration (at the project site) is given by

$$A = 2000 e^{0.8M} R^{-2}$$

Table 15.1. Modified Mercalli Scale, 1956 version (Richter 1958 in Hunt 1984).

Intensity	Effects
I	Not felt. Marginal and long period effects of large earthquakes.
II	Felt by persons at rest, on upper floors, or favorably placed.
III	Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.
IV	Hanging objects swing. Vibration like passing of heavy trucks or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV wood walls and frames creak.
V	Felt outdoors, duration estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.
VI	Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc. off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken (visibly, or heard to rustle – CFR).
VII	Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices (also unbraced parapets and architectural ornaments – CFR). Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.
VIII	Steering of motor cars affected. Damage to masonry C, partial collapse. Some damage to masonry B, none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.
IX	General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. (General damage to foundations – CFR). Frame structures, if not bolted, shifted off foundations. Frames cracked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.
X	Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beacheads and flat land. Rails bent slightly.
XI	Rails bent greatly. Underground pipelines completely out of service.
XII	Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.

Note: Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering (which has no connection with the conventional Class A, B, C construction).

– Masonry A: Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.

– Masonry B: Good workmanship and mortar; reinforced, but not designed to resist lateral forces.

– Masonry C: Ordinary workmanship and mortar; no extreme weaknesses such as non-tied-in corners, but masonry is neither reinforced nor designed against horizontal forces.

– Masonry D: Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

– CFR indicates additions to classification system by Richter (1958).

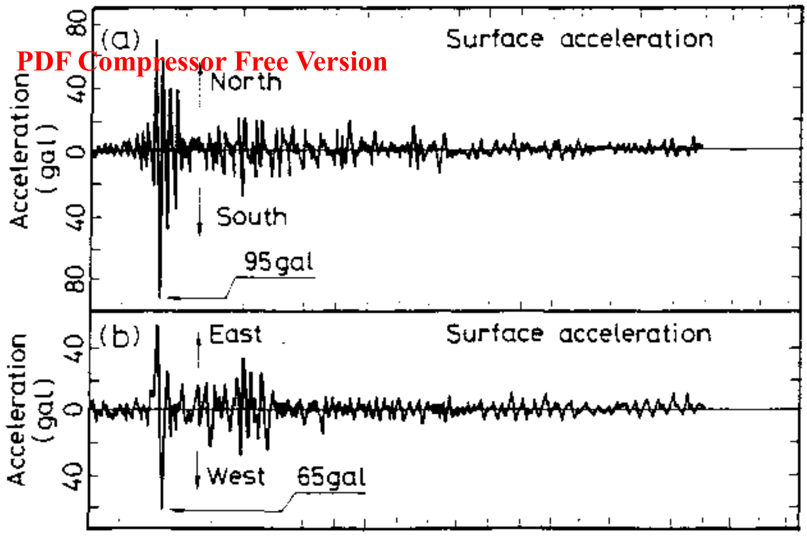


Figure 15.1. Earthquake accelerograph (USNRC 1985).

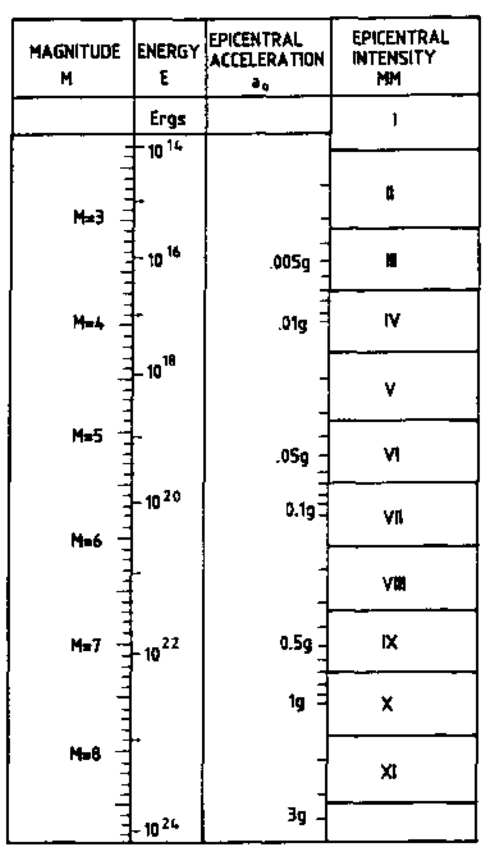


Figure 15.2. Approximate correlation between magnitude, released energy, epicentral intensity and acceleration (adapted from Hunt 1984).

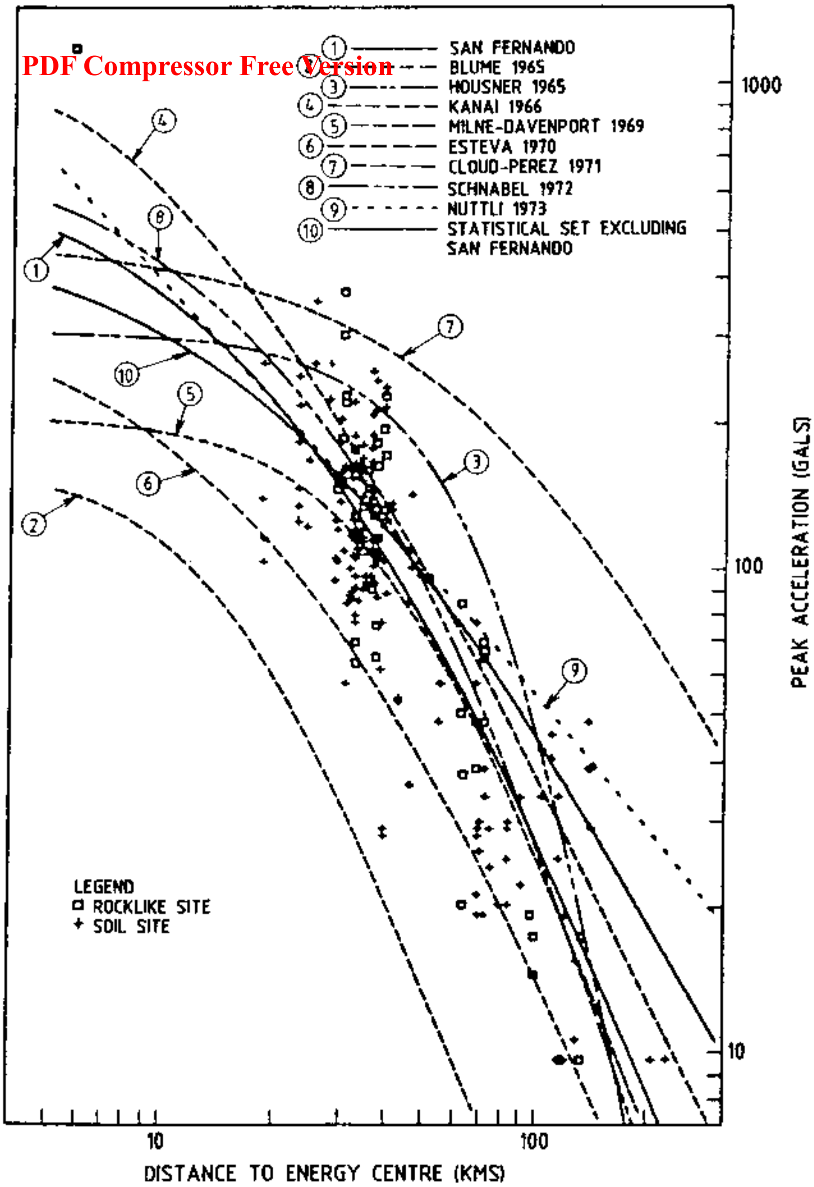


Figure 15.3. Attenuation equations for magnitude 6.5 (compared to data from strong motion stations recording February 9, 1971, San Fernando earthquake; Donovan & Bornstein 1978).

where A = peak acceleration as % of acceleration due to gravity
R = focal distance in km
M = earthquake magnitude.

The generalised form of this equation is

$$y = b_1\, e^{b_2 M}\, (x)^{-b_3}$$

where y = attenuated motion value (acceleration, velocity, etc.)
x = focal distance

M = magnitude
b_1, b_2, b_3 are constant attenuation coefficients.

There have been many different formulae developed for different parts of the world. Figure 15.3, reproduced from Donovan & Bornstein (1978), shows how variable the response and predictions can be.

Hunt (1984) also gives a relationship for intensity:

$$I_s = C_1 + C_2 I_o - C_3 \ln [R + C]$$

where I_s = site intensity (Modified Mercalli)
I_o = epicentre intensity (see Figure 15.2)
R = focal distance km
C, C1, C2, C3 = constants
ln [R + C] = error term.

15.2.2.2 *Amplification*

The attenuation equations discussed above, allow estimation of peak bedrock acceleration at a site.

As discussed by Martin (1988), sites underlain by stiff soil and thick cohesionless soil will experience similar peak accelerations to the 'bedrock' values. However, in sites underlain by soft or medium soft clays, or shallow cohesionless soils, the maximum ground acceleration may be different to bedrock accelerations. Martin (1988) indicates that for ground accelerations less than 0.1 g amplification may occur, whereas for very high accelerations (e.g. > 0.3 g) the peak accelerations may be attenuated due to energy dissipation. He quotes the example of the 1985 Mexico earthquake where bedrock accelerations were 0.03 g, and peak accelerations of 0.17 g (at a reduced frequency) were recorded at the surface of soft soil areas.

Topographic effects are also important. An accelerograph sited on a ridge high on a dam abutment, is likely to record significantly higher accelerations than one sited at the river level, so care must be taken in siting such instruments. An appreciation of the extent of magnification due to topographic effects can be gained by comparison with the crest and base accelerations for a dam as discussed in Section 15.4.1.

15.2.3 *Design earthquakes*

15.2.3.1 *General definitions*

ICOLD (1983) recommend the use of two different design earthquakes.

Design Basis Earthquake (DBE). The earthquake which is liable to occur at least once during the expected life of the structure (also called operating basis earthquake, OBE). The expected life is likely to be 100 years or more, and is not necessarily the life assumed for calculating the economics of the scheme. The DBE will usually be obtained by probabilistic analysis of recorded earthquakes.

Maximum Credible Earthquake (MCE). The maximum earthquake event that can be conceived to affect the dam, taking into consideration the presence of potentially active faults in the vicinity of the dam.

ICOLD (1983) indicate that for embankment dams, the MCE should not cause the dam: '(a) to fail due to liquefaction of material in the dam or its foundations; (b) to collapse due to movement at a slip surface in the slope or through the foundation; (c) to lose its freeboard; (d) to develop uncontrolled leakage through cracks or at interfaces with structures or abutments; (e) spillways and hydraulic controls to be damaged to the extent that dangerous conditions develop.'

It is also indicated that: 'It is very difficult to define what should be limiting conditions when an embankment dam is subjected to the DBE. Some yield and permanent distortion is probable, and can be accepted. Although this has happened, the strength of the dam is not significantly impaired. The application of the design earthquake to the dam in the design process serves only to provide an estimate of possible movement. The associated structures and hydraulic controls should remain operable.'

ICOLD (1989c) introduced the concept of Maximum Design Earthquake which will usually be the MCE but may be of smaller magnitude.

15.2.3.2 *Estimation of design basis earthquake*

The DBE is usually estimated by probabilistic risk analysis of earthquakes, recorded in the vicinity of the dam site. ICOLD (1989c) recommend using procedures such as the Cornell-McGuire method (Cornell 1968, McGuire 1976). The procedure is:

– obtain records of earthquake magnitude, epicentral location and focal depth;

– determine attenuation laws to allow estimation of peak acceleration at the dam site for each recorded earthquake. This may be based on accelerograph records, or by using one or more of the published attenuation equations. The advice of seismologists should be sought in selecting attenuation equations;

– plot the estimated peak accelerations for the period of record, and extrapolate to the required return period for the DBE.

Figure 15.4 shows the results of such an analysis for a dam in the Solomon Islands. In this case there were no accelerograph records, so published attenuation equations were used. The wide scatter of results is apparent as is the general trend to predict larger accelerations using later published equations.

US National Research Council (1985) describe an alternative approach by Liao (1985), which might be applied if earthquake activity is known to be related to faults which can be identified. The principles are summarized in Figure 15.5.

This approach would seem to have little application in other than a few well instrumented areas.

Lo & Klohn (1990) indicate that for tailings dams the DBE (or Operating Basis Earthquake, OBE) is often chosen as the earthquake which has a 10% probability of exceedance in a 50 year period, or an annual probability of exceedance of 1 in 475.

Finn et al. (1990), in his keynote speech to the ICOLD/ANCOLD International Symposium on Safety and Rehabilitation of Tailings Dams, indicated that recent evidence obtained by trenching across active faults was that probabilistic methods may underestimate the probability of larger seismic events. He also emphasised the importance of accurate historical information in assessing earthquakes.

15.2.3.3 *Estimation of maximum credible earthquake*

In a similar way to estimation of probable maximum precipitation, the maximum credible earthquake should be estimated using a deterministic rather than probabilistic approach.

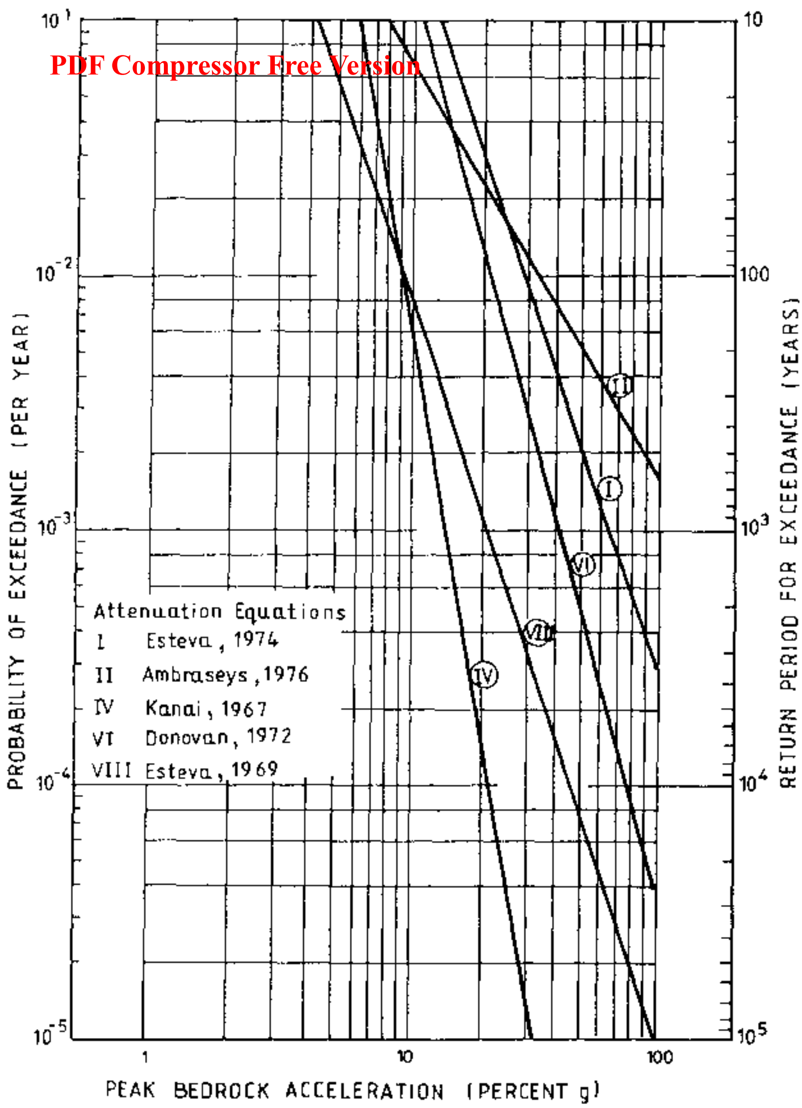


Figure 15.4. Peak bedrock acceleration vs probability of exceedance, Lungga Dam Site, Solomon Islands (Coffey & Partners 1981a).

The procedure will usually involve:

– Identification of major faults within the vicinity of the dam. This may involve an area up to several hundred kilometres from the site.

– Assessment of whether the faults are active or potentially active, by consideration of whether modern earthquakes have been recorded along the fault. This may involve geomorphological studies, e.g. of displaced river terraces; and/or trenching across faults to identify the ages of displacement.

– Assessment of the Maximum Credible Earthquake on each identified fault. This will

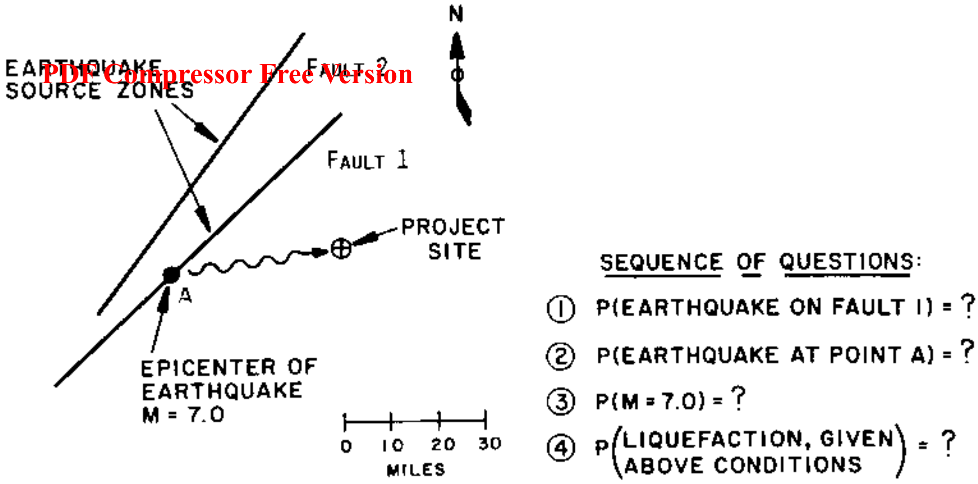


Figure 15.5. Probabilistic approach to estimating liquefaction (USNRC 1985).

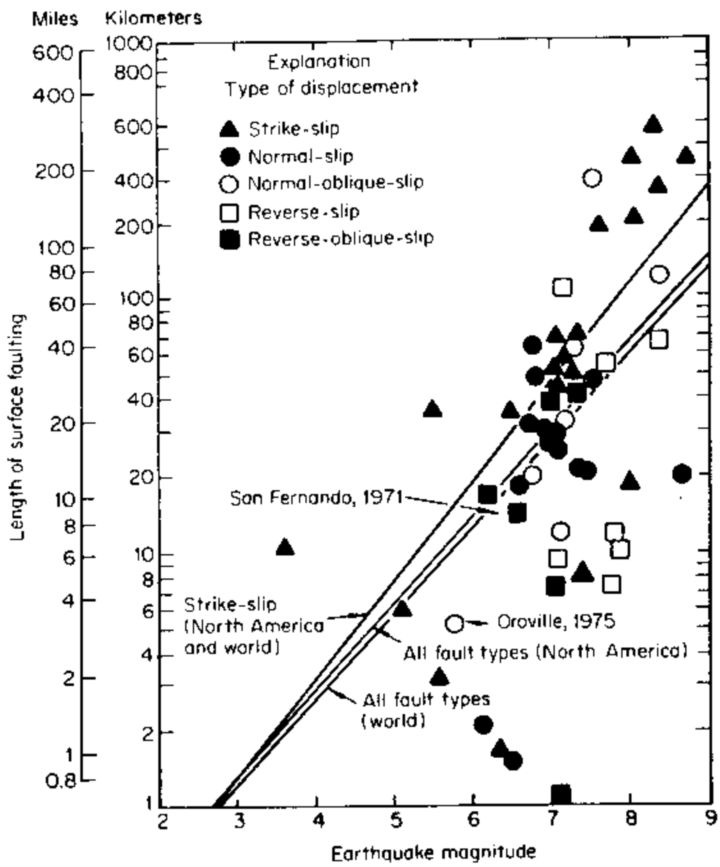


Figure 15.6. Scatter diagram of length of surface faulting related to earthquake magnitude from historical events of surface faulting throughout the world. Lines are least-square fits (Taylor & Cluff 1977 in Hunt 1984).

usually be determined by considering the length of the fault but also the general seismicity of the area. Figure 15.6 shows a plot of length of fault vs earthquake magnitude which may be used as a general guide.

– Assessment of the maximum acceleration at the dam site resulting from the MCE for each of the faults, and determination of the most critical earthquake. Since the duration of the earthquake, and the period of oscillations are dependent on the magnitude of the earthquake and the effect on the dam is dependent on these factors as well as maximum accelerations, more than one earthquake may have to be used in analysis.

In the event that it is not practicable to estimate the MCE from the above procedure, a probabilistic approach may be used, but with caution. An annual probability of exceedance of the order of 1 in 10 000 should be used in these circumstances (Lo & Klohn 1990, ICOLD 1989c).

When a dynamic analysis is to be carried out for the DBE or MCE, a seismograph of the earthquake must be produced using the calculated peak acceleration and records of earthquakes at the site or at sites with similar geological conditions and causative earthquakes.

15.2.3.4 *Earthquakes induced by the reservoir*

As discussed in ICOLD (1983) and ICOLD (1989c), there is documented evidence to prove that impounding of a reservoir sometimes results in an increase of earthquake activity at or near the reservoir. Guha et al. (1980) summarize the known cases up to that time.

ICOLD (1983) conclude that:

– earthquakes of magnitude 5 to 6.5 were induced in 11 of 64 recorded events

– the greatest seismic events have been associated with very large reservoirs (but there is insufficient data to show any definite correlation between reservoir size and depth and seismic activity)

– in view of the above, a study of possible induced seismic activity should be made at least in cases where the reservoir exceeds $10^9 m^3$ in volume, or 100 m in depth

– the load of the reservoir is not the significant factor, rather it is the increased pore water pressure in faults, leading to a reduction in shear strength over already stressed faults.

ICOLD (1983) and (1989c) give more details and references on this issue.

15.3 LIQUEFACTION OF DAM EMBANKMENTS AND FOUNDATIONS

One of the most critical issues relating to the effect of earthquakes on dams is whether liquefaction of the dam or the dam foundation may occur and, if so, what the consequences would be. Much of the research of dynamic analysis methods has been driven by the need to investigate potential liquefaction. The following outlines some of the general concepts relating to liquefaction.

15.3.1 *Liquefaction, flow failure and deformation failure – Definitions*

The USNRC (1985) gives the following definitions of liquefaction and related phenomena: 'Liquefaction is the phenomenon where excessive deformations or movement occur as a result of transient or repeated disturbance of saturated cohesionless soils.'

This will be accompanied by an increase in pore pressure, and partial or total loss of shear strength.

Flow failures describe the condition where the soil mass deforms continuously under a shear stress equal to the static shear stress applied to it, e.g. slope instability, total bearing capacity failure.

Deformation failures involve large permanent displacement or settlement, but the earth mass remains stable without great changes of geometry. Flow failures and deformation failures are forms of liquefaction.

It is emphasised that in most cases, on deforming, the liquefied soil does retain a residual undrained shear strength.

Phenomena which occur as a result of liquefaction include:

– Soil boils: Formed by water flowing upward to the surface from a zone of (earthquake induced) high pore pressure. Soil may flow with the water.

– Flow failures of the slopes of a dam, or of soil on slopes generally, due to reduction in strength in the soil.

– Lateral spreads: Blocks of soil may move laterally on the liquefied soil on gentler slopes (e.g. 0.3 to 3 degrees).

– Ground oscillation: Causes fissures to form between blocks of soil oscillating on a liquefied layer.

– Loss of bearing capacity (because shear strength is reduced).

– Buoyant rise of buried structures such as tanks and pipelines.

– Ground settlement.

– Retaining wall failures.

15.3.2 *Soils susceptible to liquefaction*

Saturated sands, silty sands and gravelly sands are susceptible to liquefaction. Figures 15.7 and 15.8, reproduced from USNRC (1985), show the particle size envelopes for potentially liquefiable natural soils, and mine tailings.

It can be seen that the mine tailings are more susceptible to liquefaction than natural soils, possibly reflecting their uniform size and recent deposition. Troncoso (1990) and Troncoso et

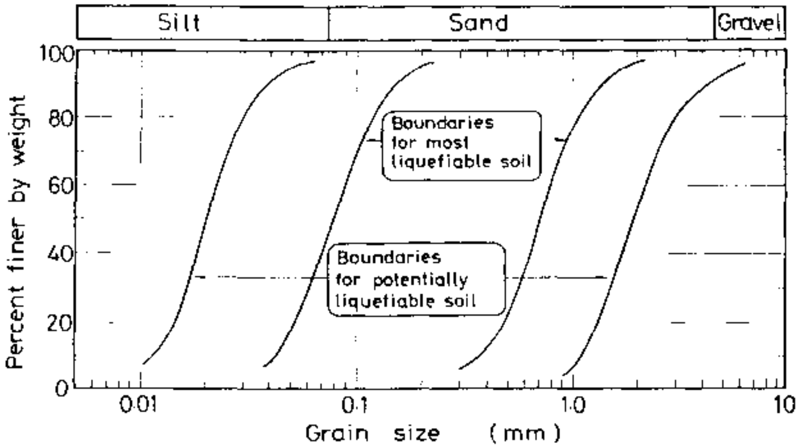


Figure 15.7. Limits in the gradation curves separating liquefiable and unliquefiable soils (Tsuchida 1970, USNRC 1985).

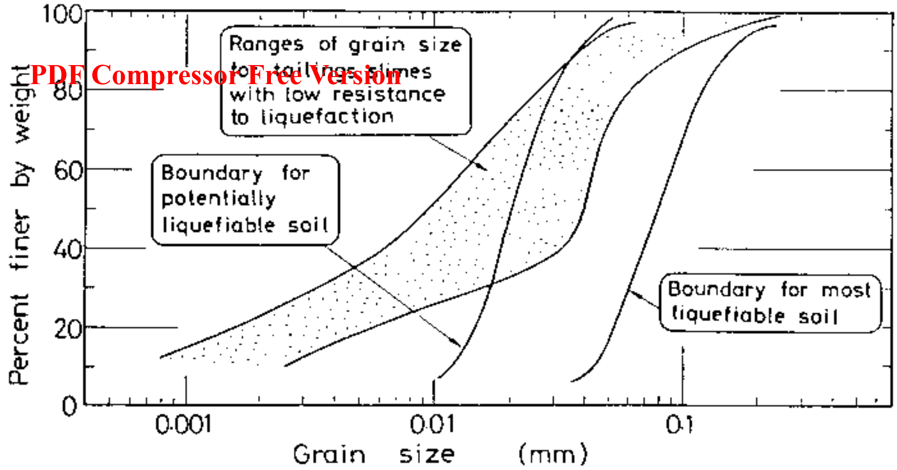


Figure 15.8. Ranges of grain sizes for mine tailings slimes with low resistance to liquefaction (Ishihara 1985, USNRC 1985).

al. (1988) present some evidence that tailings will 'age' and develop greater resistance to liquefaction with time.

15.3.3 *Liquefaction – General concepts*

Extensive laboratory testing has been carried out by many researchers to explain the phenomenon of liquefaction. These show that:

a) Cyclic loading causes densification of dry granular soils by particle rearrangement due to the back and forth straining. If, however, the soil is saturated and not allowed to drain during cyclic loading, the decreases in volume cannot occur and the tendency to decrease volume is counteracted by an increase in pore pressure and decrease in effective stress – see Figure 15.9.

Hence, soil starting at A and subject to cyclic loading which would otherwise have ended at B, will in fact have stresses represented by C where total stress σ_0 is taken by σ'_f and Δ_u, i.e. the pore pressures must increase to maintain equilibrium in this undrained condition.

The pore pressures build up gradually with the number of cycles of loading, and only if the pore pressures build up to equal the total stress does the 'initial liquefaction' (effective stress $\sigma' = 0$) condition occur.

This is shown in Figure 15.10 for loose and dense sand subjected to cyclic triaxial testing.

b) The number of cycles (shearing of the soil) to reach the $\sigma' = 0$ condition depends on the relative density and the magnitude of the cyclic stress compared to initial stress τ_c/σ'_{vo}) where τ_c = cyclic shear stress and σ'_{vo} = vertical stress.

It can be seen that loose sands are more susceptible to liquefaction than dense sands. There are also indications that cyclic shear strains must exceed a threshold value (about 0.01%) before any pore pressure buildup occurs.

c) The stress path followed by the soil affects liquefaction potential. Figure 15.13 shows the p-q stress paths during a cyclic triaxial test, the effective stress path moving from cycle 1 to the left, before reaching the failure envelope on the k_f line in cycle 21.

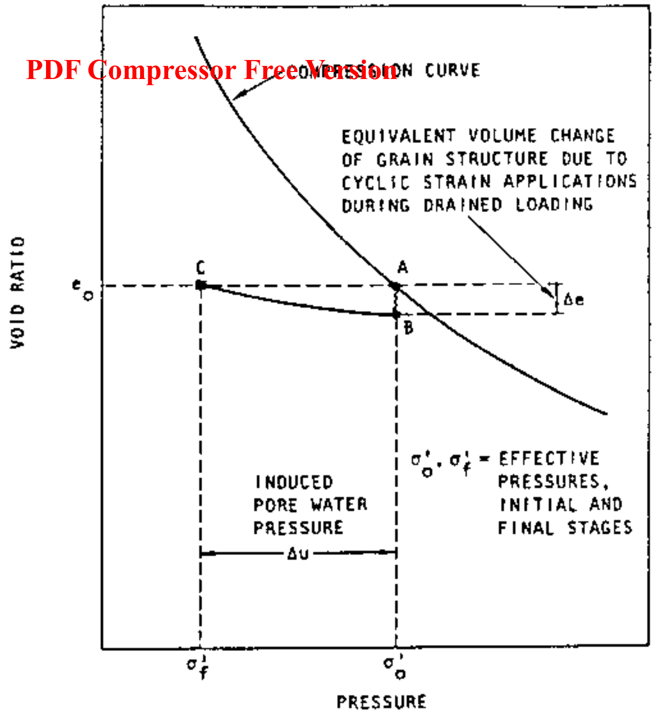


Figure 15.9. Schematic illustration of mechanism of pore pressure generation during cyclic loading (Seed & Idriss 1982, USNRC 1985).

Once a failure line is reached, pore pressures and strain development accelerates. After $\sigma' = 0$ is reached the stress paths are up and down the failure lines passing through or near the origin twice in each cycle as shown.

If the sample is subjected to anisotropic consolidation, or to a constant shear stress over and above the cycled stress, the stress paths are altered significantly, as shown in Figure 15.14.

Here the cycling takes the sample from the failure line (compression side) away to a non-failure condition, so while the soil will continue to strain, the continuous $\sigma' = 0$ condition is not reached.

d) 'Liquefied' soils still exhibit a residual undrained strength, even if the $\sigma' = 0$ condition develops during cycling. Once cycling stops the soil will still exhibit shear strength, e.g. for medium dense and dense sands the undrained steady state strength at large strains as shown in Figure 15.15b is not affected.

However, if as in Figure 15.15a, for loose sand, the cyclic loading strains the soil past its monotonic (static) loading peak shear strength, the remaining shear strength may not be sufficient to sustain the static loading τ_d, leading to further strain, and loss of shear strength until S_{us} is reached. This can result in a flow failure.

Figure 15.16 shows conditions where there is a static shear stress (τ_d) and if type (a) conditions in Figure 15.15 apply, flow failure will occur.

e) *In situ* stress conditions affect the liquefaction potential. Martin (1988) cites Vaid et al. (1979), Chang et al. (1983) and Ishabashi et al. (1985) as showing that the presence of an initial (or static) shear stress increases the cyclic stress ratio required to increase pore pressures (e.g. to $\sigma' = 0$ conditions), when compared to tests without initial shear stresses. Hence the worst

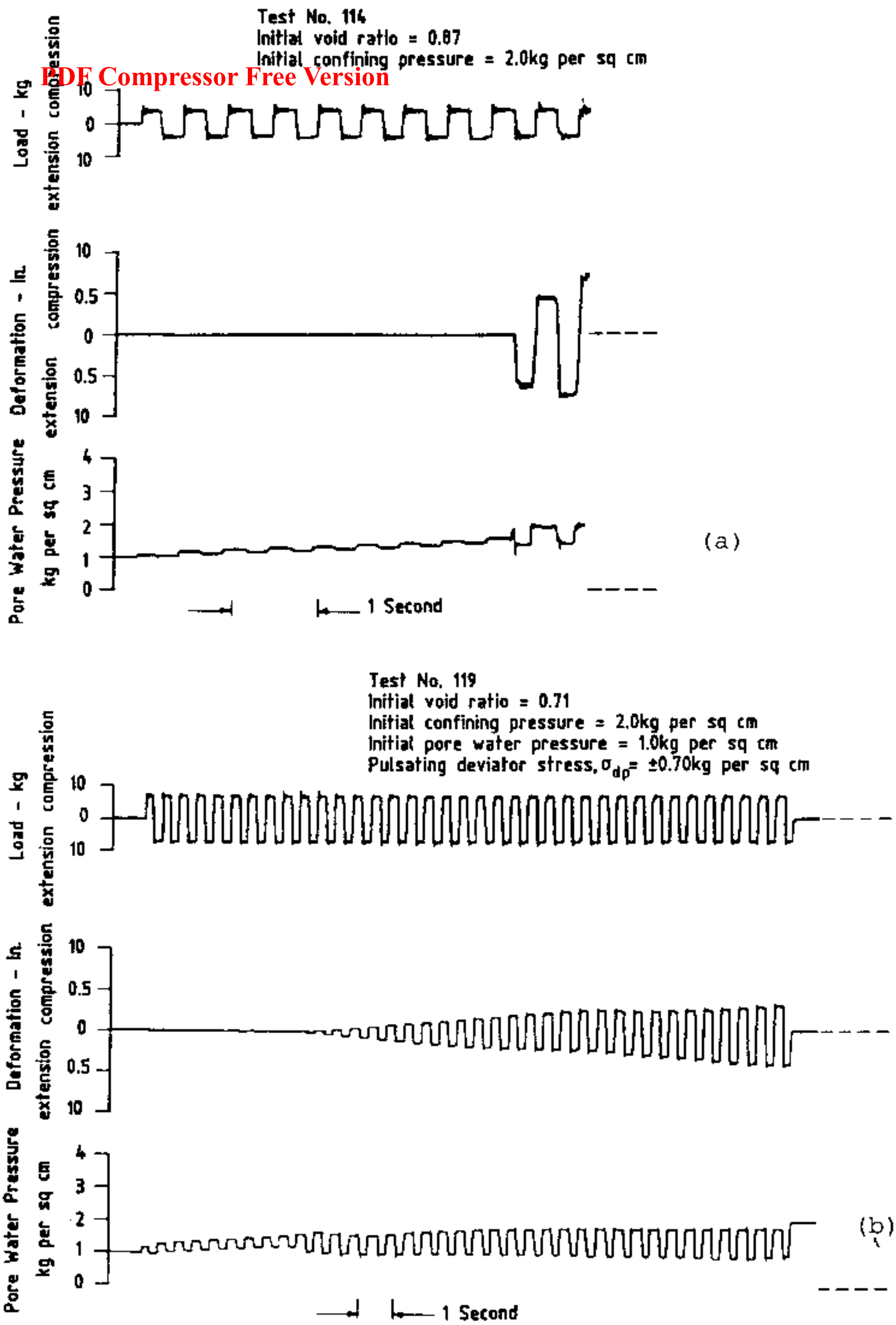


Figure 15.10. Results from isotropically consolidated cyclic triaxial tests. a) loose sand, b) dense sand (Seed & Lee 1966, USNRC 1985).

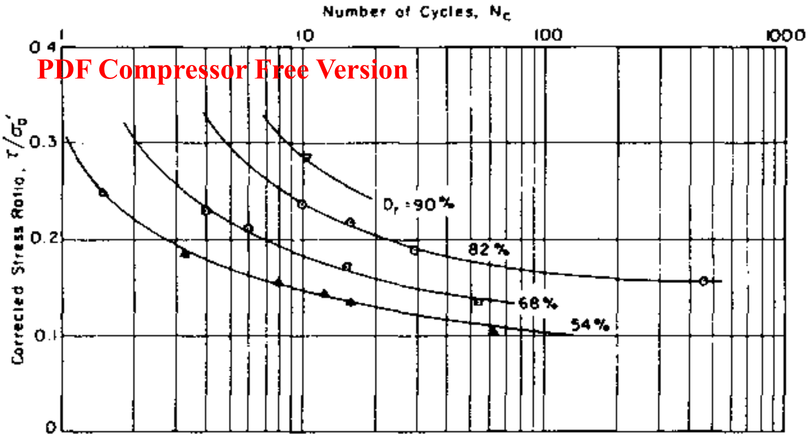


Figure 15.11. Stress ratio τ_o/σ'_{vo} versus number of cycles to initial liquefaction from tests on a shaking table (De Alba et al. 1976, USNRC 1985).

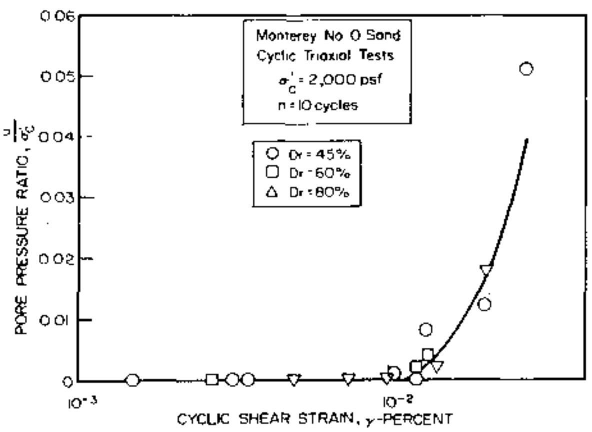


Figure 15.12. Pore pressure ratio versus cyclic shear strain, illustrating the concept of a threshold strain required to cause generation of excess pore water pressures (Dobry et al. 1981, USNRC 1985).

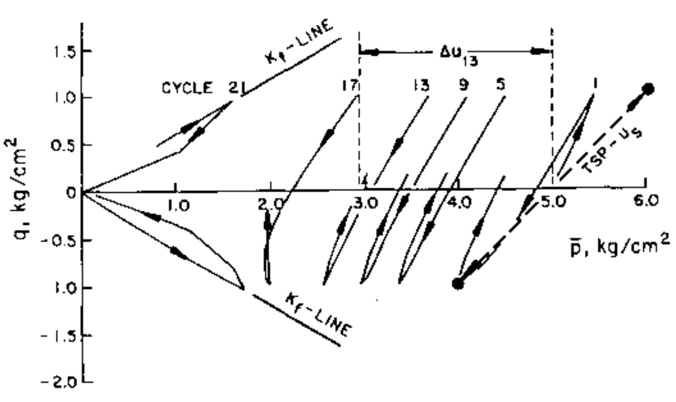


Figure 15.13. Effective and total stress paths for isotropically consolidated fine sand (relative density = 55 percent) in a triaxial test (Hedberg 1977, USNRC 1985).

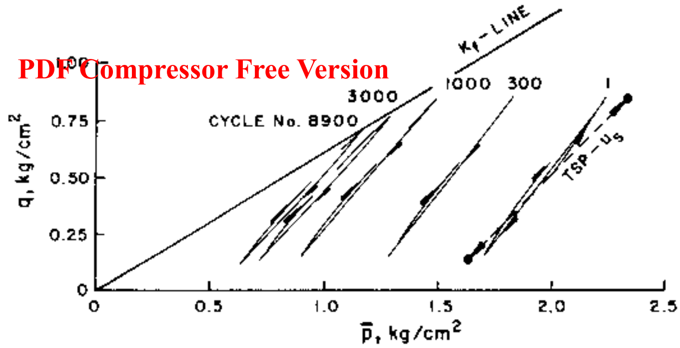


Figure 15.14. Effective and total stress paths during cyclic triaxial compression test on fine sand (relative density = 42 percent) (Hedberg 1977, USNRC 1985).

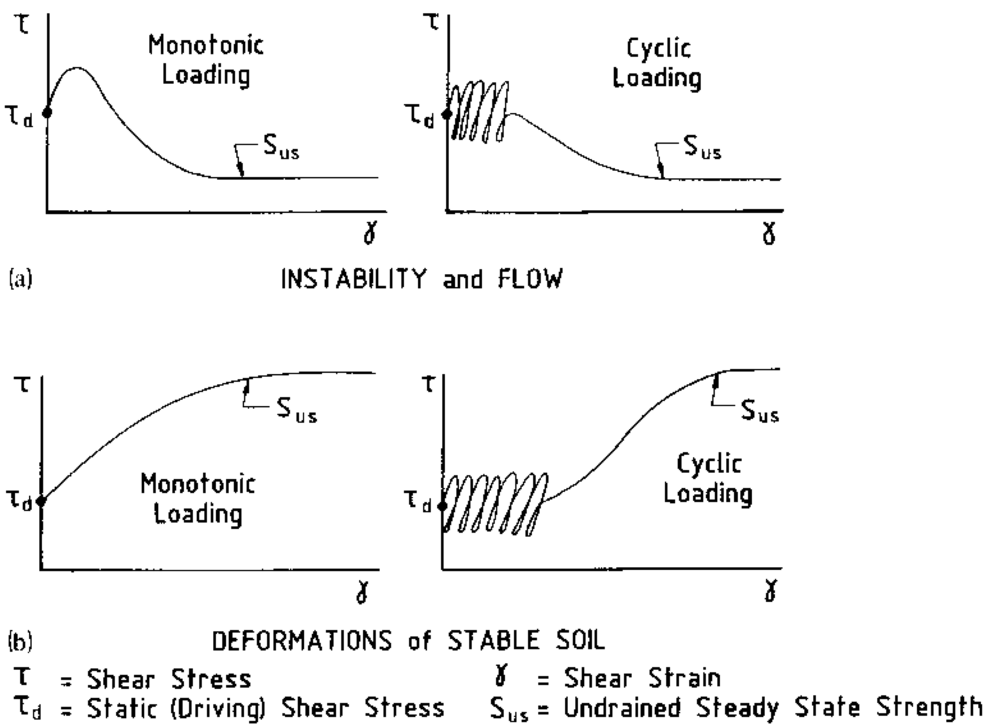


Figure 15.15. a) Unstable and b) Stable behavior under static and cyclic loading (Castro 1976, USNRC 1985).

condition is the horizontal or near horizontal saturated ground condition. Martin (1988) produced Figure 15.17 (based on work by Dean & Schofield 1983) to show this effect in a model dam. Point A, which has low initial shear stress develops greater pore pressure under cyclic loading, than points D, C and B which have higher initial static shearing stresses.

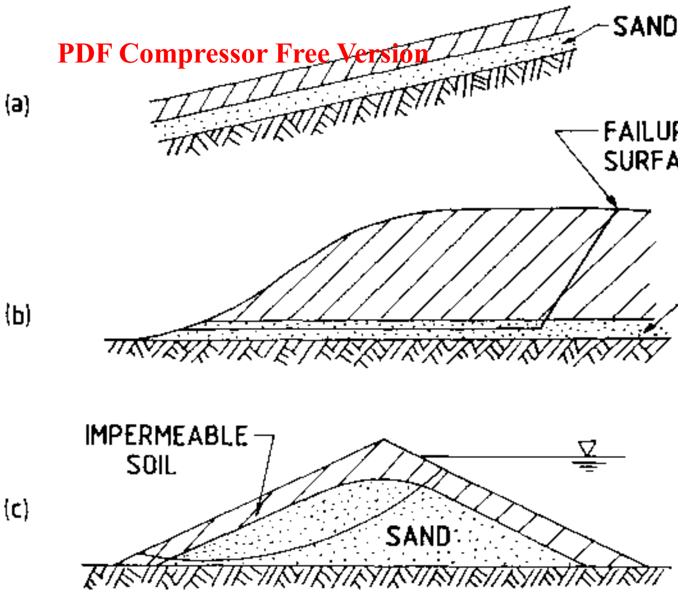


Figure 15.16. Examples of situations involving the existence of significant shear stress in the soil: a) sloping ground; b) embankment on level ground; c) earth dam.

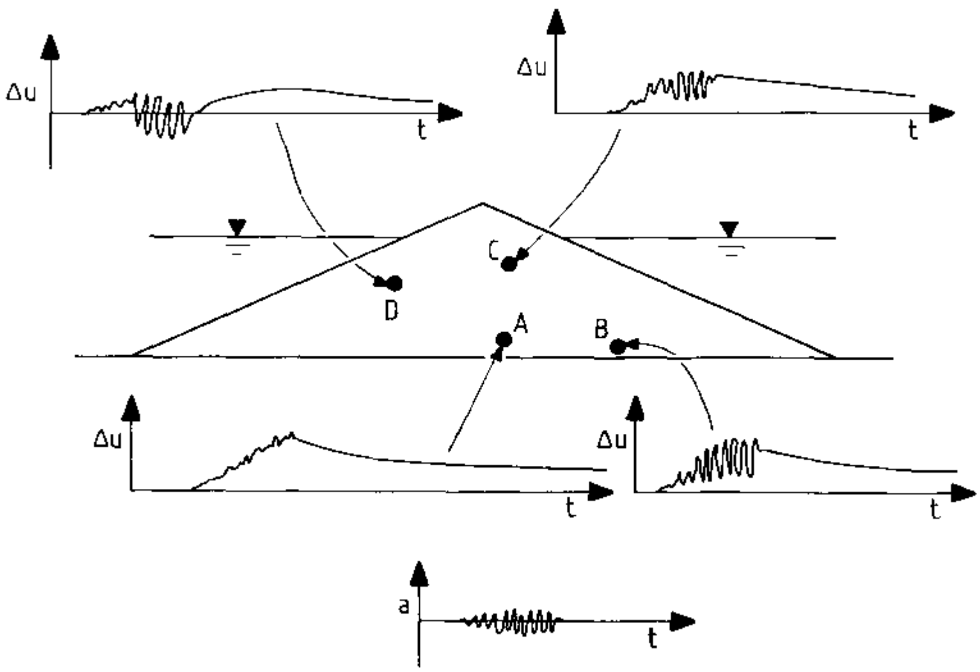


Figure 15.17. Pore pressure measured in a model dam shaken in a centrifuge (Martin 1988, Dean & Schofield 1983).

15.4 EVALUATION OF LIQUEFACTION POTENTIAL

15.4.1 *Empirical approach of Seed et al. for horizontal ground*

The simplest and most practical method of assessing whether there is a potential for liquefaction for horizontal ground conditions, has been developed by Seed and his co-researchers over many years. The method is semi-empirical, and is based on the maximum acceleration induced by the earthquake a_{max}, the SPT 'N' value corrected for the SPT hammer energy and for overburden pressure [$(N_1)_{60}$], earthquake magnitude (M), and fines content of the soil (% passing 75 μm). It is based on recorded cases of liquefaction during earthquakes in USA, Japan and China. Details are given in Seed & De Alba (1986) and in USNRC (1985).

The steps are:

a) Estimate a_{max} for the site, i.e. maximum acceleration at the ground surface.

b) Estimate average peak shear stress (τ_{av}) induced by the earthquake from

$$\tau_{av} / \sigma'_o = 0.65\, a_{max}\, \sigma_o\, r_d / \sigma'_o\, g$$

where τ_{av} = average peak shear stress

σ'_o = effective overburden stress at depth under consideration

σ_o = total overburden stress at the same depth

r_d = stress reduction factor; = 1 at ground surface; = 0.9 at 11 m

c) Determine the stress ratio (τ_{av}/σ'_o) which will lead to liquefaction, by the following procedure: 1) Determine $(N_1)_{60}$, from the measured SPT 'N' value, corrected to 60% energy ratio from Table 15.2 and to 100 kPa overburden stress using Figure 15.18.
i.e.

$$N_1 = C_N\, N_m$$

and

$$N_{60} = N_m \cdot \frac{ER_m}{60}$$

Table 15.2. Summary of energy ratios for SPT procedures (Seed & De Alba 1986).

Country	Hammer type	Hammer release	Estimated rod energy (%)	Correction factor for 60% rod energy
Japan	Donut	Free-fall	78	78/60 = 1.30
	Donut	Rope and pulley with special throw release	67	67/60 = 1.12
United States	Safety	Rope and pulley	60	60/60 = 1.00
	Donut	Rope and pulley	45	45/60 = 0.75
Argentina	Donut	Rope and pulley	45	45/60 = 0.75
China	Donut	Free-fall	60	60/60 = 1.00
	Donut	Rope and pulley	50	50/60 = 0.83

(a) Japanese SPT results have additional correction for borehole diameter and frequency effects.
(b) Prevalent method in the United States today.
(c) Pilcon-type hammers develop an energy ratio of about 60%.

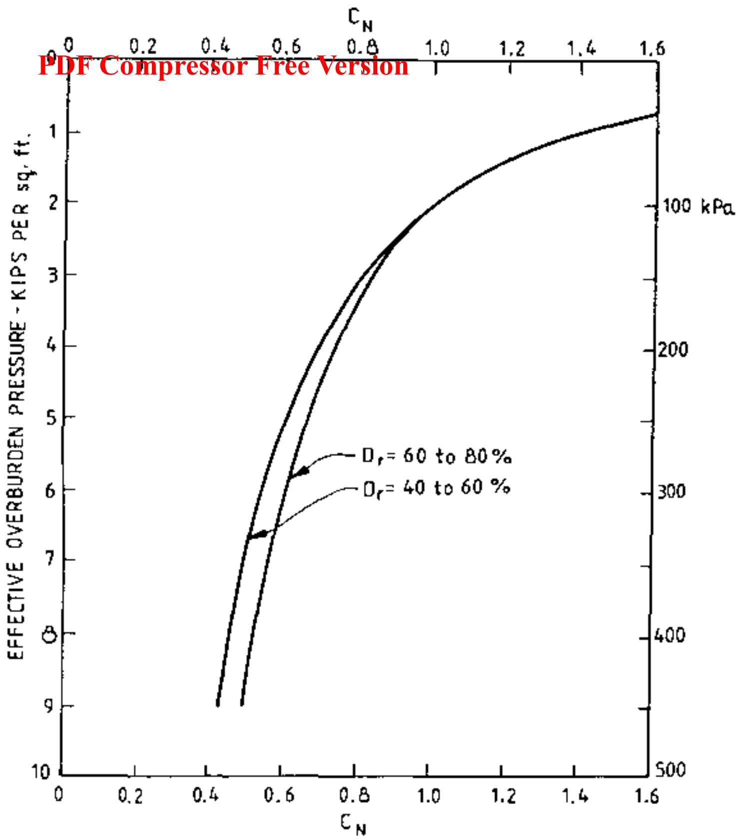


Figure 15.18. Graphs for value of overburden correction factor C_N (Seed & De Alba 1986).

where N_m = measured SPT value
ER_m = measured rod energy.

2) Determine τ_{av}/σ'_o) which will lead to liquefaction for a magnitude 7.5 earthquake (from Fig. 15.19). For earthquakes of magnitude other than 7.5, correct the values of τ_{av}/σ'_o) by the factors in Table 15.3.

d) Compare the estimated τ_{av}/σ'_o for the design earthquake with that required to cause liquefaction.

The 'factor of safety' to be applied to this depends on the degree of conservatism in selecting a_{max} and may be between 1 and 1.3 (on τ_{av}/σ'_o).

e) Figure 15.19 applies to sand with less than 5% fines passing 75 μm. Figure 15.20 applies where there are more fines. Note that the fines reduce susceptibility to liquefaction. Fines in this context are cohesionless (silt).

This approach has been extended to use of the Static Cone Penetration test (CPT) by correcting CPT q_c values to SPT $(N)_{60}$ using Figure 15.21. Alternatively Figure 15.22 can be used in lieu of Figure 15.20.

There are several problems in applying the Seed et al. semi empirical approach:

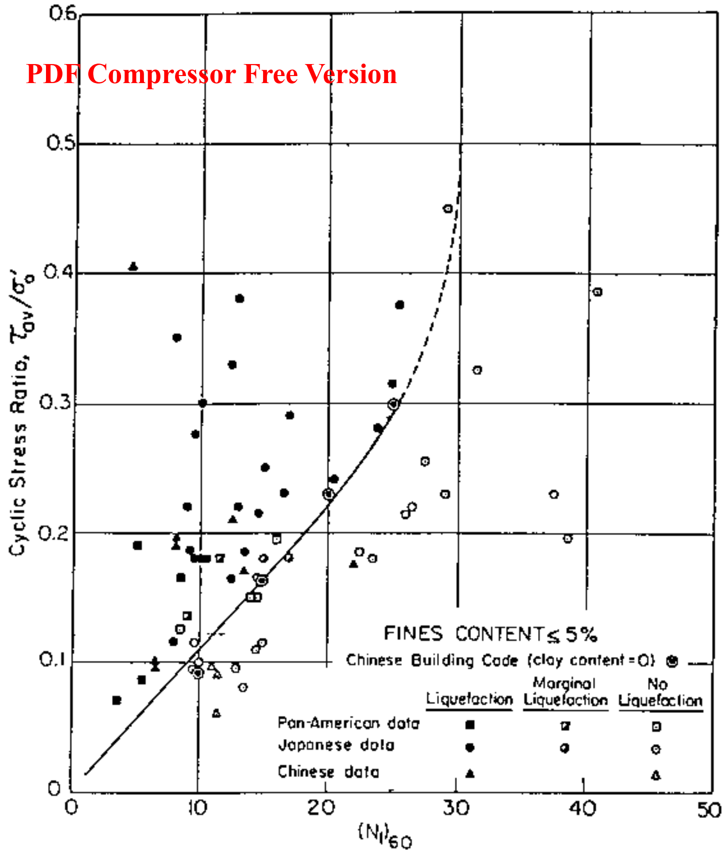


Figure 15.19. Relationship between stress ratios causing liquefaction and $(N_1)_{60}$ values for clean sands for magnitude 7.5 earthquakes (Seed & De Alba 1986).

Table 15.3. Representative number of cycles and corresponding correction factors (Seed & De Alba 1986).

Earthquake magnitude (M)	Number of representative cycles at 0.65 τ_{max}	Factor to correct abscissa of curve in Figure 15.19
8.5	26	0.89
7.5	15	1.00
6.75	10	1.13
6.0	5-6	1.32
5.25	2-3	1.5

a) It has been developed for level or near level ground conditions. For most dam applications it can therefore only be directly applied to assess whether liquefaction would occur without the dam.

Seed & Harder (1990) recommend that to allow for the static driving stress due to the dam, a correction factor should be applied to the calculated τ_{av}/σ'_o. This is calculated using

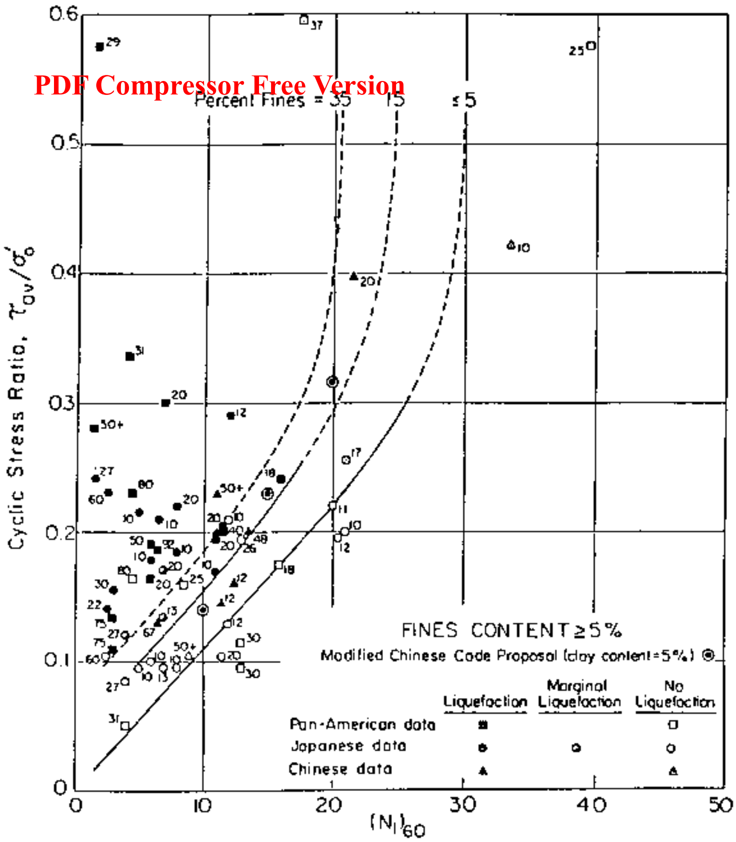


Figure 15.20. Relationship between stress ratios causing liquefaction and $(N_1)_{60}$ values for silty sands for magnitude 7.5 earthquakes (Seed & De Alba 1986).

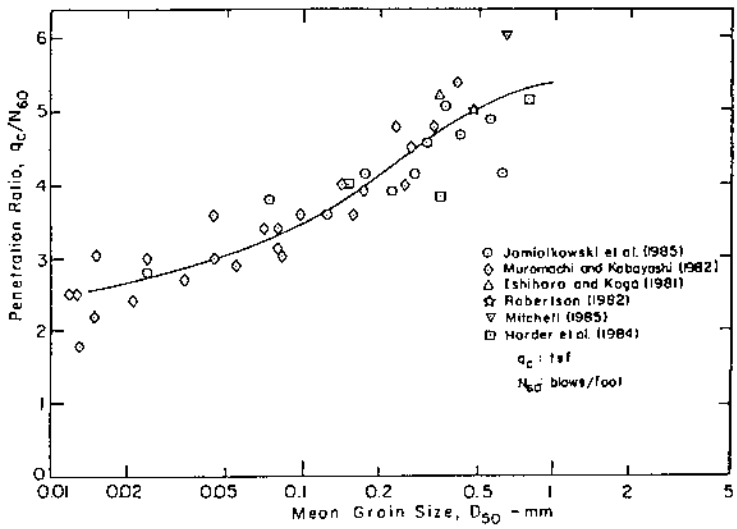


Figure 15.21. Variation of q_c/N_{60} ratio with mean grain size (q_c measured in tsf ≈100 kPa) (Seed & De Alba 1986).

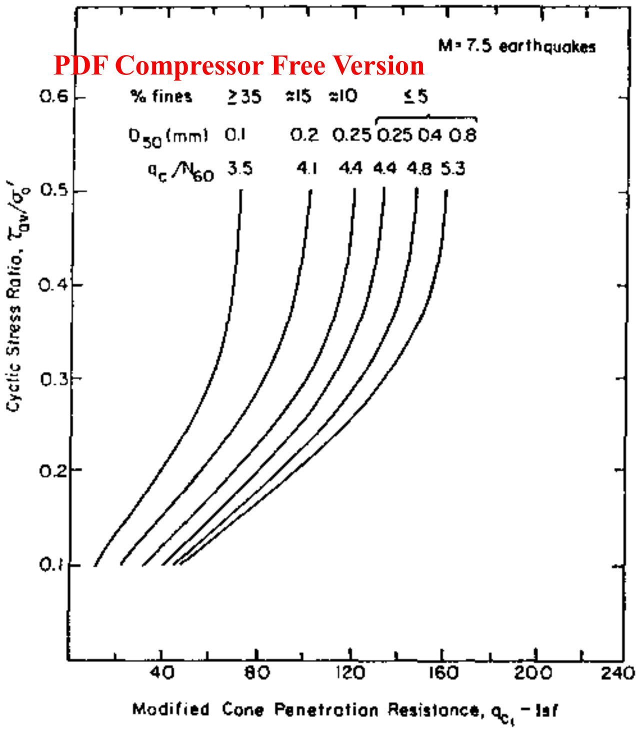


Figure 15.22. Relationship between stress ratio causing liquefaction and cone tip resistance for sands and silty sands (1 tsf ≈ 100 kPa) (Seed & De Alba 1986).

$$(\tau_{av} / \sigma'_o)_{\alpha=\alpha} = (\tau_{av} / \sigma'_o)_{\alpha=o} \cdot K_\alpha$$

where K_α is a correction factor determined from the Figure 15.23. To determine K_α, the relative density and α(the ratio of static driving shear stress on a horizontal plane to the initial effective overburden stress, τ_{hv}/σ'_o).

The correction only applies to soils where $\sigma'_o \leq 300$ kPa. At higher initial effective stresses, soils will be less dilative or more contractive, and lower K_α values will apply.

It will be noted that for soils with a relative density greater than 45%, K_α is greater than unity, so the effect of the *in situ* shear stress is to lessen the likelihood of liquefaction. This agrees with the indications from tests shown in Figures 15.13 and 15.14, that a soil with superimposed shear stress is less likely to liquefy than the same soil with no superimposed shear stress.

b) Seed & Harder (1990) also recommend that for effective overburden stresses greater than 100 kPa, a further correction is required to the calculated τ_{av}/σ'_o. This is calculated using

$$(\tau_{av} / \sigma'_o)_{\sigma'_o=\sigma'_o} = (\tau_{av} / \sigma'_o)_{\sigma'_o=100\text{kPa}} \cdot K_\sigma$$

where K_σ is a correction factor determined from σ'_o and Figure 15.24.

It will be noted that there is considerable scatter in the data points above and below the design curve. At a symposium held conjointly with the Ninth Panamerican Conference on Soil Mechanics and Foundation Engineering in Vina del Mar, Chile, August 1991, several speakers

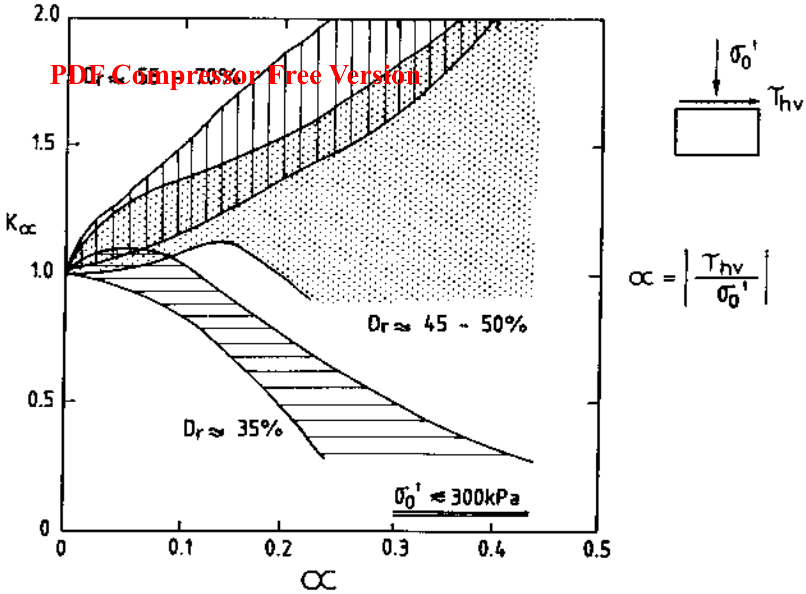


Figure 15.23. Correction factor K_α to account for static driving shear stresses (Seed & Harder 1990).

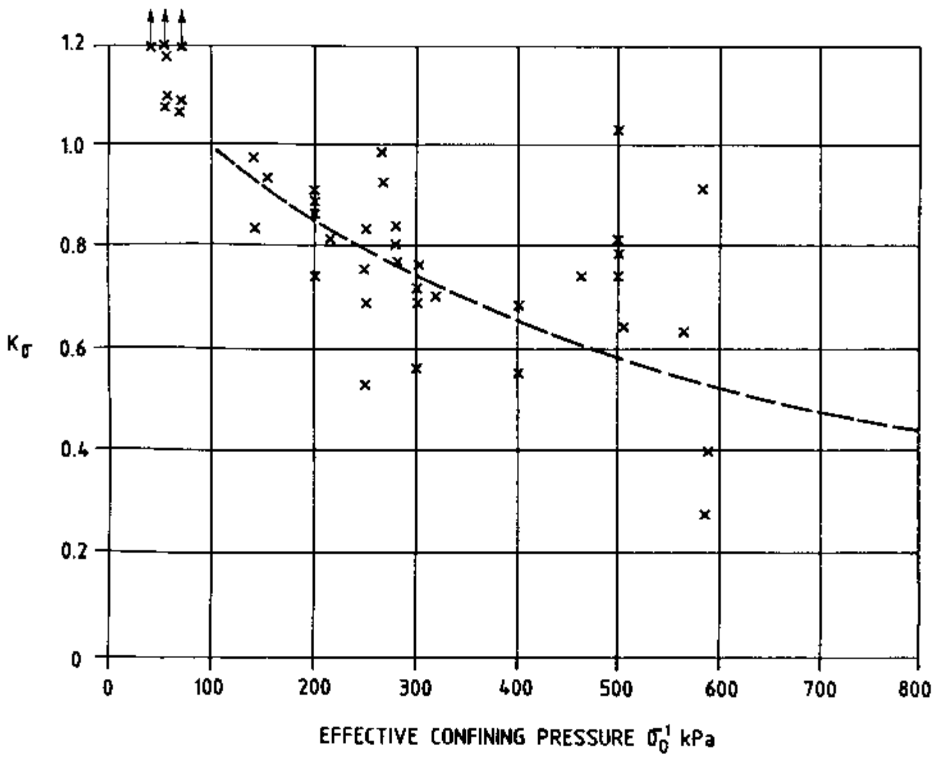


Figure 15.24. Correction factor K_σ to account for in-situ effective stress σ'_0 greater than 100kPa (Seed & Harder 1990).

voiced concern with the apparently conservative results yielded when this correction factor is applied.

c) In most natural soil deposits the SPT values are variable (over that increase which occurs with the increasing overburden stresses, and above the values which would be expected due to overburden stress). There are no clear guidelines given by Seed et al. as to how this should be accounted for. It is the opinion of the authors, that where there is some continuity of low SPT values in the strata, and good drilling and SPT testing practice has been followed, then individual layers of low liquefaction resistance must be assumed to be present.

15.4.2 *Use of shear wave velocity (V_s) to assess liquefaction for horizontal ground*

Shear wave velocity is affected by many of the variables which influence liquefaction, e.g. (relative) density, confining pressures, stress history, geologic age. Hence, it has some promise as an indicator of liquefaction.

The shear wave velocity may be obtained by downhole, crosshole or surface to downhole seismic methods, or by a seismic cone penetration test (a modification of the piezocone test) developed by Campanella et al. (1986).

One possible approach is to develop a 'data bank' relating liquefaction to shear wave velocity, as was done for SPT.

Figure 15.25 presents some data from two earthquakes. However, the 'data bank' is limited and more results would be needed before the method could be used in practice.

A second approach using shear wave velocity is to estimate the peak strain caused by the earthquake and compare this to the threshold strain which causes liquefaction (shown to be about 0.01%, see Figure 15.12).

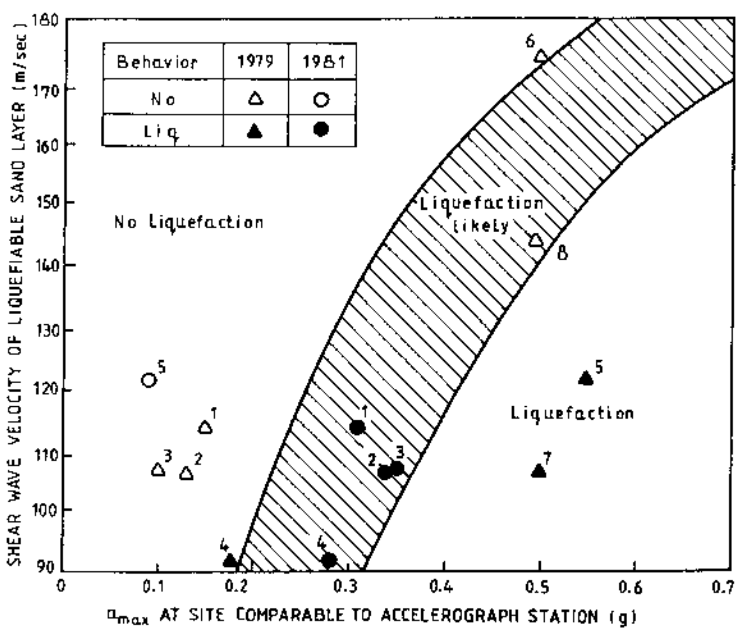


Figure 15.25. Influence of shear wave velocity on liquefaction potential of sands from field and parametric studies, Imperial Valley, California. M = 5.5 to 6.5 (Bierschwale & Stokoe 1984, USNRC 1985).

$$\gamma = \frac{\tau}{G}$$

$$= \frac{(a_{max}/g)\,\sigma_o\, r_d}{G}$$

where γ = shear strain
τ = peak shear stress
G = shear modulus
a_{max} = peak acceleration at ground surface
g = acceleration due to gravity
σ_o = total overburden stress
r_d = stress reduction factor (0.92-1.0).

Since $G = \rho V_s^2$

If the soil density is constant with depth

$$\gamma = \frac{a_{max}\, z\, r_d}{(G/G_{max})_\gamma V_s^2}$$

where $(G/G_{max})_\gamma$ = modulus reduction factor for strain
≈ 0.8 in many cases
V_s = shear wave velocity
z = depth.

So assuming an average value for r_d,

$$\gamma = 1.2\ a\ z/V_s^2$$

Hence, by measuring V_s as a function of depth by downhole/crosshole techniques, γ can then be calculated and compared to the critical strain, typically 0.01%.

USNRC (1985) indicate that this will usually give a conservative evaluation of liquefaction potential.

These methods, based on shear wave velocity, give an alternative to those based on SPT and CPT. They are particularly useful in gravelly sands, or interbedded sand and gravel/sand soils, where the SPT and CPT values can be influenced by the presence of gravel.

Shear wave seismic investigations are no longer an oddity, and in view of the usefulness of shear waves in estimating the shear modulus G for dynamic analysis, such approaches can be expected to be used more in the future.

15.5 ANALYSIS OF STABILITY AND DEFORMATIONS

15.5.1 *Pseudo static limit equilibrium methods*

15.5.1.1 *Uniform acceleration coefficients*

The earliest attempts at including the effects of earthquake on the analysis of the stability of dams was the pseudo static approach. In this method, a horizontal force is applied to each slice (in limit equilibrium analysis), equal to the weight of the slice multiplied by the seismic coefficient K. The seismic coefficient is generally taken as being uniform for the whole of the dam, and less than the estimated maximum horizontal acceleration at the base of the dam in recognition of the fact that the accelerations are of short duration and cyclic. Vertical accelerations are usually neglected in the analysis. ICOLD (1986c) indicate that 'good results are obtained for preliminary designs – provided a series of empirical design coefficients is inserted.' Reference is made to Japanese practice as outlined in Baha & Watanake (1979). US Corps of Engineers (1984b) recommend use of a seismic coefficient equal to one-half the predicted peak bedrock acceleration. They also recommend use of undrained strenghts in clays, drained strenghts in granular materials. They reduce strenghts by multiplying by 0.8, and use a minimum factor of safety of 1.0. The method is used only for initial 'screening.'

15.5.1.2 *Non uniform acceleration coefficients*

It has been shown by analysis and monitoring of dams, that the earthquake motion at the base of the dam is usually magnified, giving accelerations at the dam crest which may be several times greater than at the base. This is discussed in Martin (1988), who cites Martin (1965), Makdesi & Seed (1978) and Abdel-Ghaffar & Scott (1981), and concludes that for practical purposes the average curves proposed by Makdesi & Seed (1978), shown in Figure 15.26, can be used to assess the effect of these different accelerations rather than carrying out shear slice or finite element analysis.

When such amplified accelerations are imposed into a limit equilibrium analysis a more realistic factor of safety can be obtained, but this is an instantaneous figure. Often it will be less than unity, so 'failure' will occur. Newmark (1965), and later Sarma (1973; 1979) and Makdesi & Seed (1978), developed a simplified method for estimating the deformations which would result from the cumulative period in an earthquake when the acceleration exceeds the 'yield

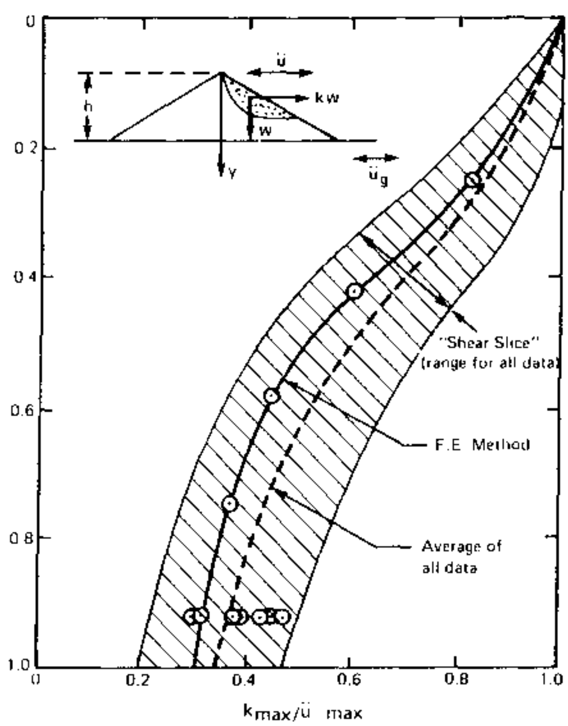


Figure 15.26. Variation of seismic coefficient K_{max} with depth of the base of the potential slide mass (Makdesi & Seed, 1978).

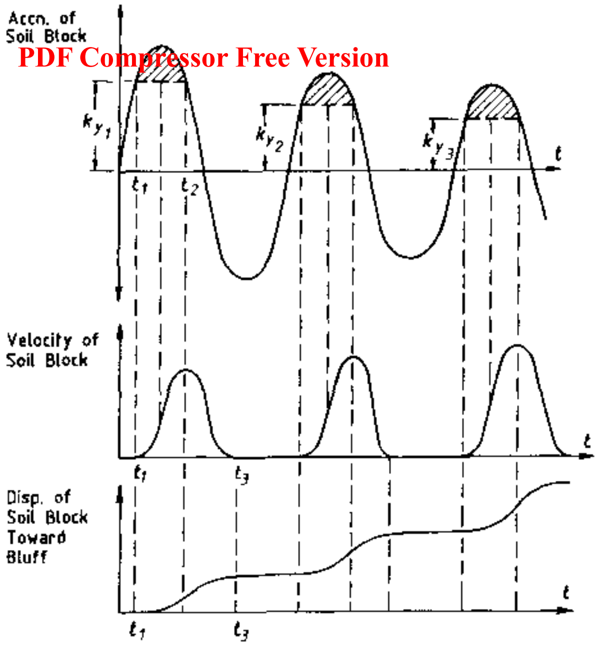


Figure 15.27. Integration of accelerograms to determine downslope displacements (Martin 1988).

acceleration.`Downslope movements are then calculated using a double integration approach, as shown in Figure 15.27.

These methods fail to properly model the strain dependent nature of the soil during an earthquake and generation of pore pressures which may occur under the cyclic loading.

As indicated by ICOLD (1986c) and Seed (1979), the methods are only applicable in design of dams in which the material strength (including pore pressures) is not appreciably affected by the cyclic loading – such as well compacted clay, clayey sand, dense sand or rockfill.

Seed et al. (1985) discuss the use of such an approach for concrete face rockfill dams. In this case the dynamic response accelerations were estimated by dynamic finite element analysis, in which the non linear stress strain characteristics of the dam are taken into account by the use of an equivalent linear representation of the soil, involving use of strain dependent modulus and damping properties for the soils in the embankment. The shear modulus is determined by

$$G = 1000\ k_2\ (\sigma'_m)^{0.5}$$

where G = effective shear modulus at a given strain level

σ'_m = effective mean principal stress at any point

k_2 = soil modulus coefficient.

The maximum value of $k_2 = k_{2max}$ occurs at low strains (10^{-4}%) and ≈ 120 to 170 for compacted gravels and rockfills.

The results of a typical analysis of the response of a rockfill dam are shown in Figure 15.28a. In this case the dam under consideration was a sloping core dam subjected to base motions, representative of the effects of a Magnitude 8.25 earthquake occurring near the dam site. The maximum acceleration of the base motions was thus 0.5 g and the duration of the shaking was 80 seconds. The value of K_{2max} for the main sand-gravel-cobble fill in the embankment was

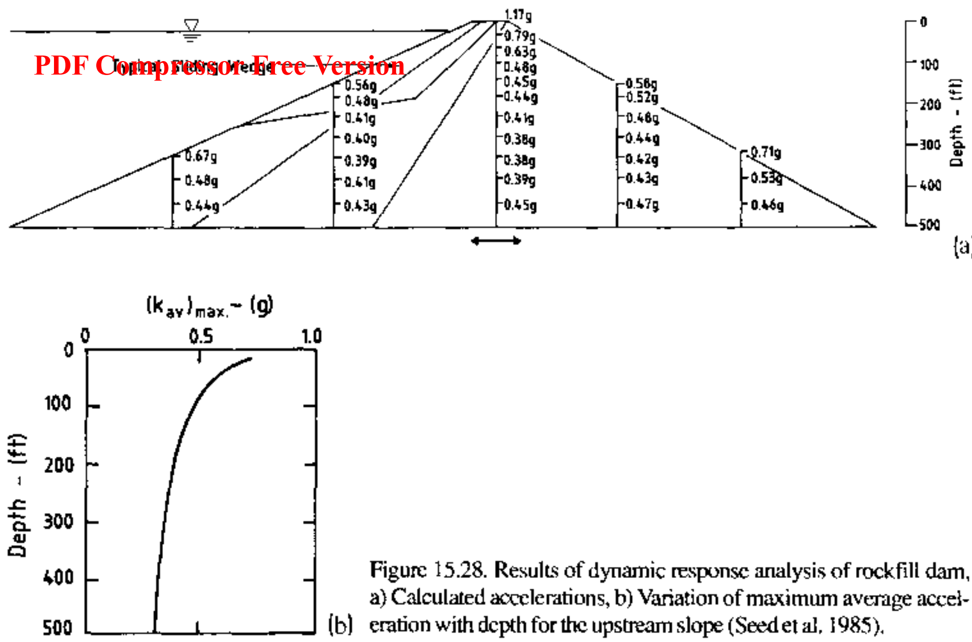


Figure 15.28. Results of dynamic response analysis of rockfill dam, a) Calculated accelerations, b) Variation of maximum average acceleration with depth for the upstream slope (Seed et al. 1985).

estimated to be 170. The accelerations induced in the dam by the shaking are shown in this figure. The variation of the maximum average acceleration, $(K_{av})_{max}$, acting on wedges of the embankment extending to different depths below the crest of the embankment, are shown in Figure 15.28b.

The earthquake-induced deformations of such a dam are usually computed by a type of analysis procedure, originally proposed by Newmark (1965), and subsequently modified to take into account the complexities associated with deformation analyses of earth structures, as discussed in Seed (1979).

The method requires the determination of the average time-history of accelerations acting on any given wedge, expressed by the maximum value of this acceleration, $(K_{av})_{max}$, and the level of acceleration at which deformation begins to occur for that wedge, expressed by the yield acceleration, k_y. The magnitude of the deformations can be estimated from a knowledge of the ratio $k_y/(K_{av})_{max}$ for the wedge under consideration, and the earthquake magnitude, M, as shown in Figure 15.29. Values of $(K_{av})_{max}$ are determined from the dynamic response analysis, as shown in Figure 15.29, and the values of k_y from the strength characteristics of the soils involved and the geometry of the wedge under consideration.

For a wedge of the type shown in Figure 15.30, on the downstream side of the embankment, analyses show that deformations are negligible, even for the high levels of accelerations developed in this embankment, and that movements are likely to be associated only with wedges extending to limited depths below the surface of the embankment.

For vertical faced wedges, the yield acceleration is determined from:

$$k_y = \sin(\phi' - \alpha)$$

where ϕ' = angle of friction of rockfill
α = slope of the base of the wedge.

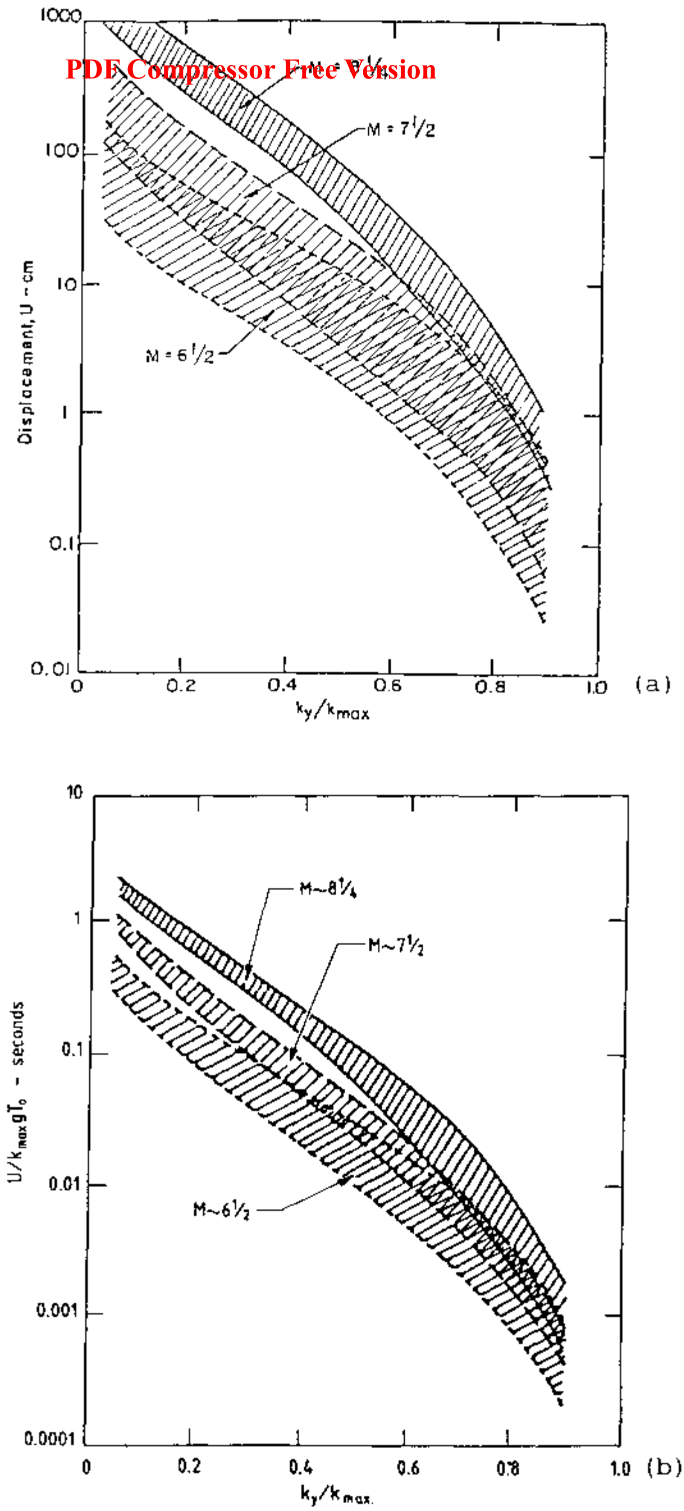


Figure 15.29. a) Variation of permanent displacement with yield acceleration, Seed et al. (1985). b) Variation of yield acceleration with normalized permanent displacement, Makdisi & Seed (1978).

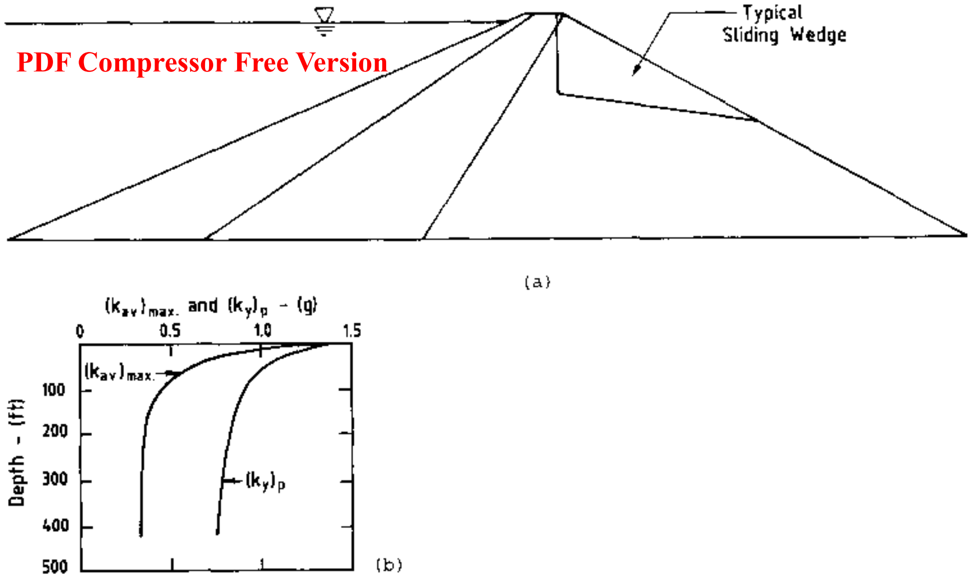


Figure 15.30. Variation of $(k_{av})_{max}$ and $(k_y)_p$ with depth for wedges on the downstream slope (Seed et al. 1985).

Allowance is made for the markedly nonlinear Mohr envelope for rockfill, with significantly higher effective friction angle at low confining stress than at higher confining stress (values of $\phi' = 57°$ at 2 psi, 50° at 20 psi and 41° at 500 psi were used).

Deformations are then calculated using Figure 15.29b.

Hence, for a shallow slice of rockfill near the downstream face, such as that in Figure 15.30:

average downstream pressure = 5 psi (35 kPa)

$\phi' \approx 54°$

$\alpha \approx 1{:}1.8$ or $29°$

$k_y \approx \sin(54° - 29°) = 0.42$

$(k_{av})_{max} \approx 0.8$

and

$k_y/(k_{av})_{max} \approx 0.53.$

So from Figure 15.29a, for a

M7.5 earthquake displacement ≈20 cm

and for a

M8.5 earthquake displacement ≈60 cm.

Seed et al. (1985) indicate that the estimates so derived are likely to be conservative.

Seed et al. go on to discuss the design slopes for concrete face rockfill dams for earthquakes, using the above analysis approach, and recommend that the slopes of the dam should be flattened to limit displacements. They argue that:

If for areas with magnitudes less than 6.5, and maximum ground accelerations less than 0.3 g, dam slopes are 1 in 1.35 to 1 in 1.5, then, (a) for areas with earthquakes magnitude greater than 6.5 and up to 7.5, and ground accelerations up to 0.5 g, slopes should be flattened to about 1 to

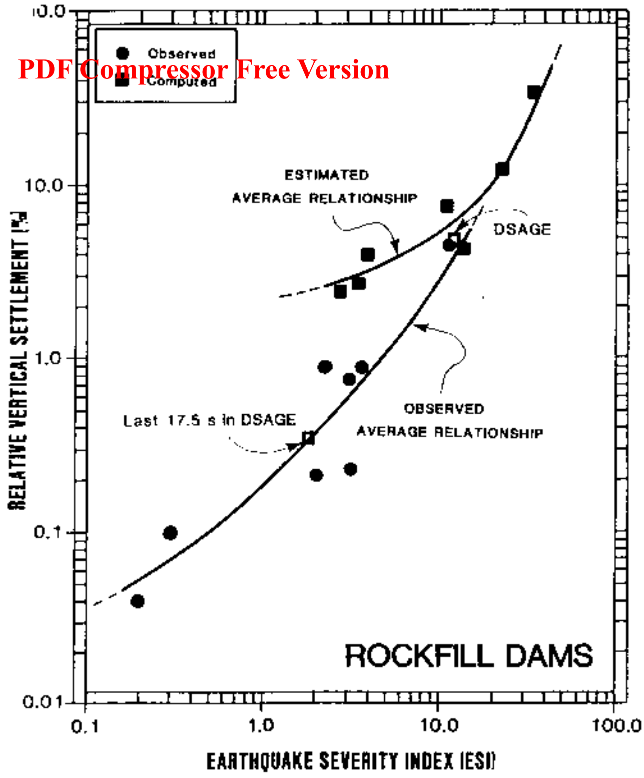


Figure 15.31. Relationships between crest settlement and Earthquake Severity Index (Bureau et al. 1985).

1.65,and (b) for areas with earthquakes magnitude greater than 7.5 and up to 8.5, with ground accelerations up to 0.5 g, slopes should be flattened to about 1 to 1.8.

They emphasise that these are only general guidelines, and each case must be taken on its merit. They suggest these flatter slopes may only need to apply to the upper part of the dam where accelerations are the greatest.

Bureau et al. (1985) review observed settlements of rockfill dams (including central core earth and rockfill) during earthquakes. They conclude that the Newmark type approach, combined with detailed dynamic response analysis, gives reasonable answers comparable to observed behaviour in rockfill dams. They suggest the use of the 'Earthquake Severity Index (ESI)' as an approximate guide to a preliminary estimate of crest settlements. The ESI is given by:

$$ESI = A(M - 4.5)^3$$

where A = peak ground acceleration (in 'g's), M = earthquake magnitude

and crest settlement, as a percentage of total dam height is estimated from Figure 15.31. They note that at low values of ESI, estimated crest settlements are higher than those actually observed after earthquakes.

15.5.1.3 *Steady state strength analysis*

Lo & Klohn (1990) describe a limit equilibrium approach which is used as a check on equi-

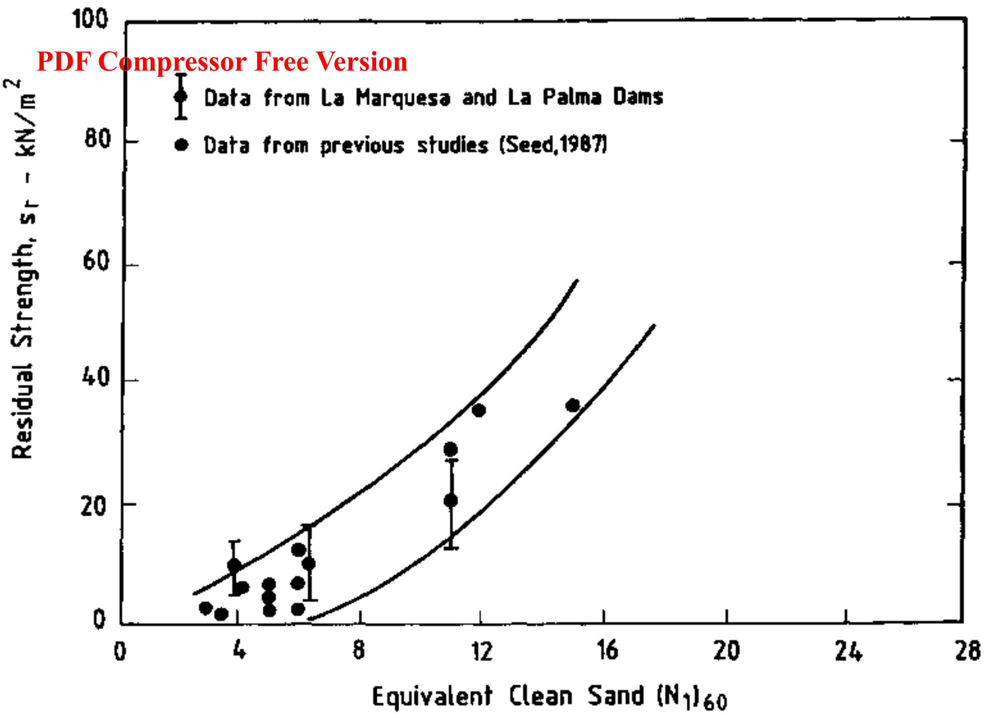


Figure 15.32. Relation between residual strength and corrected SPT 'N' value (Lo & Klohn 1990).

librium of the dam, assuming that saturated sand zones liquefy and reach what Lo and Klohn call 'steady state' strength, but which has been termed 'field residual' by Seed (1987). No seismic coefficient is included in the analysis. They suggest use of Figure 15.32 to determine the field residual undrained strength from the SPT 'N' values (corrected for efficiency and overburden stress).

This gives a quick check of the worst possible conditions. If the dam has a factor of safety greater than 1 for this severe condition, this may be sufficient to establish that more sophisticated analyses are not required. It should be noted that Figure 15.32 only applies to saturated sand zones which are assumed to liquefy. Other zones of the embankment would use undrained strengths predicted from laboratory tests (e.g. clayfill) or from the drained strength parameters (e.g. rockfill).

15.5.1.4 *Pseudo static analysis including seismically induced pore pressures*

Lo & Klohn (1990) describe a further modification to limit equilibrium analysis which was developed for tailings dams, but could in principle be used for other dams too. The method is described in Klohn et al. (1978) and adds seismically induced pore pressures to the static pore pressures in a limit equilibrium analysis. The method is incorporated in the computer program SEISLOP (Grigg & Lo 1979) and as described by Lo & Klohn (1990) includes the following steps: (1) A design earthquake and its related parameters are selected for the site. (2) Static stress conditions on an assumed failure surface are evaluated using a conventional limit equilibrium form of analysis. (3) Inertia forces are then applied to the dam, using a conventional limit equilibrium analysis to determine the resulting cyclic shear stresses on the assumed failure surface. The inertial forces are calculated using appropriate seismic coefficients originated from the elastic seismic response analyses for embankment dams by Seed & Martin (1966). (4) Using the design earthquake parameters, the physical properties of the cohesionless material, and cyclic triaxial test data, the pore pressure increases caused by the cyclic inertia forces are evaluated. These cyclic pore pressures are then added to the original pore pressures developed by seepage flow. Potential increase of pore pressure due to the additional pond force resulting from some loss of strength in tailings stored in the pond may also be added. (5) Finally, the resulting increased pore pressures are used to calculate the reduced effective normal stresses on the failure surface for the condition immediately following the last cycle of the earthquake shaking.

In discussing the method, Lo & Klohn (1990) indicate: 'The SEISLOP simplified method of analysis requires more field and laboratory data than does the limit equilibrium stability analysis using the steady state strength, and also requires the selection of a design earthquake. However, the SEISLOP analysis program is simpler and less costly to run than the programs for finite element analysis, while at the same time acknowledging the importance of energy input levels from the earthquake and the affects of seismically induced pore water pressures. It does, of course, also include many more approximations than does the finite element method.

Comparison of the results obtained, using the simplified method with actual finite element analyses, shows reasonable agreement.'

The method requires cyclic shear testing of the materials which might be subject to liquefaction, and includes the effect of sudden increase in shear stresses which are induced to the dam if the tailings liquefy. The analysis includes seismic coefficients and increased pore pressure due to the seismic loading so is more conservative than a simple pseudo static approach.

15.5.2 *Dynamic finite element analyses*

There are a number of dynamic finite element models and associated laboratory testing procedures which are used to:

a) predict the time history of stress and strain in the dam,

b) compute the generation and dissipation of pore pressures,

c) assess from the above whether 'failure,' i.e. say flow failure, will occur, or what deformations can be expected.

The subject is a complex one and it is beyond the limits of this text to discuss it in detail. The methods are summarized in ICOLD (1986c), USNRC (1985).

ICOLD (1986c) quote Seed (1979) in describing a method which uncouples (a) and (b), and assumes undrained behaviour i.e. no dissipation of pore pressures during the earthquake. The method simulates the stress-strain behaviour of the soils by varying the secant moduli with strain. The steps in the method, as described by Seed (1979) are: '(a) Determine the cross-section of the dam to be used for analysis. (b) Determine, with the cooperation of geologists and seismologists, the maximum time history of base excitation to which the dam and its foundation might be subjected. (c) Determine, as accurately as possible, the stresses existing in the embankment before the earthquake. This is probably done most effectively at the present time using finite element analysis procedures. (d) Determine the dynamic properties of the soils comprising the dam, such as shear modulus, damping characteristics, bulk modulus or Poisson's Ratio, which determine its response to dynamic excitation. Since the material characteristics are non linear, it is also necessary to determine how the properties vary with strain amplitude. (e) Compute, using an appropriate dynamic finite element analysis procedure, the stresses induced in the embankment by the selected base excitation. (f) Subject representative samples of the embankment materials to the combined effects of the initial static stresses and the superimposed dynamic stresses and determine their effects in terms of the generation of pore water pressures and the development of strains. Perform sufficient number of these tests to permit similar evaluations to be made, by interpolation, for all elements comprising the embankment. (g) From the knowledge of the pore pressures generated by the earthquake, the soil deformation characteristics and the strength characteristics, evaluate the factor of safety against failure of the embankment either during or following the earthquake. (h) If the embankment is found to be safe against failure, use the strains induced by the combined effects of static and dynamic loads to assess the overall deformations of the embankment. (i) Be sure to incorporate the requisite amount of judgement in each of steps (a) to (h) as well as in the final assessment of probable performance, being guided by a thorough knowledge of typical soil characteristics, the essential details of finite element analysis procedures, and a detailed knowledge of the past performance of embankments in other earthquakes.'

When calculating the stability against flow failure during and after the earthquake, it is assumed that the lower limiting value of the shear strength on any part of the failure surface is the steady state (or residual undrained) strength.

The undrained steady state strength (S_{us}) is sensitive to grain size distribution, particle shape and particularly the *in situ* voids ratio. Hence, tests should be carried out on undisturbed samples which are difficult to obtain.

USNRC (1985) gives the following procedure:

Step 1 – Determine *in situ* voids ratio by *in situ* testing, freezing and coring, sampling in test pits etc.

Step 2 – Determine the steady state shear strength compacted at various voids ratio (the upper line in Fig. 15.33).

Step 3 – Determine the undrained steady state strength of the undisturbed specimen by consolidated undrained triaxial test (Point A on Fig. 15.33).

Step 4 – Draw curve AB parallel to the upper line, but intercept the *in situ* voids ratio at B, and allow calculation of S_{us} *in situ*.

Vaid et al. (1990) also show that the steady state undrained strength is dependent on the stress path, with significantly lower strengths for triaxial extension tests than for triaxial compression tests.

USNRC (1985), Lo & Klohn (1990) and Finn et al. (1990), suggest that the relationship between SPT 'N' value and undrained steady state strength, shown in Figure 15.32, should be used at least as a check on the strengths obtained as described above and may be more realistic than the laboratory tests.

In either case, there is considerable margin for error in the estimated value. Methods described above which model undrained behaviour have been used extensively for dam and tailings dam design, but have limitations because they do not allow for redistribution of pore pressures, do not correctly model the 'coupled' effect of strain, pore pressure changes, and other soil properties such as dynamic modulus and damping. Methods such as those described by Finn et al. (1990), Finn (1988), Finn et al. (1987a) do allow these to be included. Finn et al. (1990) indicate that the TARA-3 program has been validated by centrifuge tests and shown to be capable of modelling acceleration, displacements and pore pressures within reasonable geotechnical engineering accuracy.

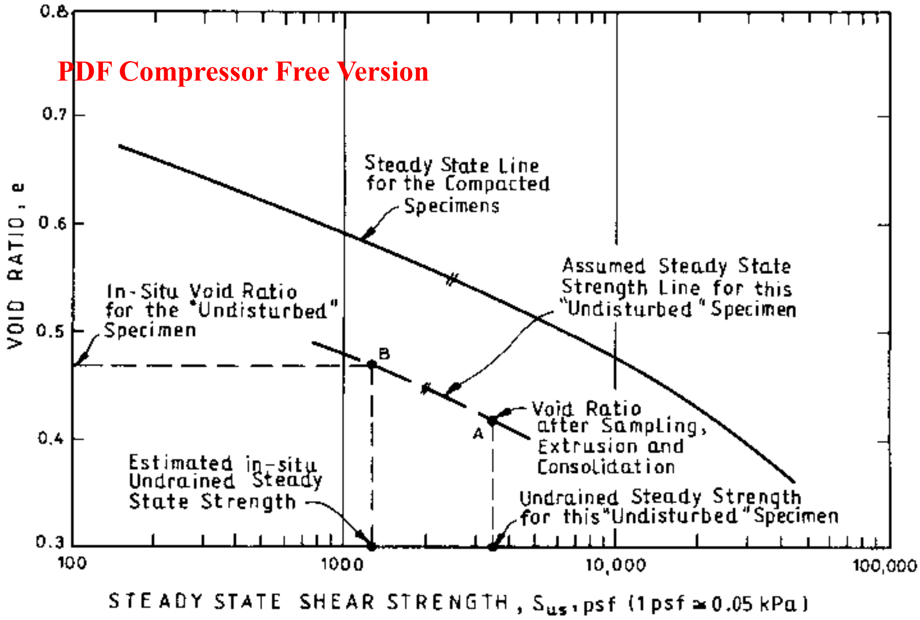


Figure 15.33. Correction procedure for measured undrained steady state strength (Poulos et al. 1985, USNRC 1985).

Finn et al. (1990) advocate the use of the TARA-3 program as a tool to assist in decision making, and do not claim a high degree of accuracy in the results. They advocate the use of parametric studies to give the designer a feel for how sensitive the answers are to variations in assumed parameters, eg. the steady state strength.

Such methods are able to model the redistribution of pore pressures which can occur during and immediately after the earthquake, and which can lead to a worse condition after the earthquake than during it. Such behaviour has been noted in actual dams (ICOLD 1986c).

It must be recognised that successful use of dynamic finite element programs involves far more than being able to run a program on a computer. The selection of input parameters and interpretation of results is far more critical and requires judgement and experience. If a dam is of sufficient size, and simple analysis shows that earthquake effects are important, it is recommended that expert advice be sought. In most cases of dams in areas of low to moderate seismicity, complicated, and often costly testing and analysis are not necessary.

15.6 DESIGN FOR EARTHQUAKE

15.6.1 *General principles*

In discussing design of dams for earthquake, ICOLD (1986c) conclude that: 'While analysis of important earth and rockfill structures is essential to avoid catastrophic failure and to determine the extent of damage which may occur due to the basic structural action, details of design are equally important. Application of commonsense measures is often all that is necessary to prevent deleterious effects. Below we quote practical recommendations taken verbatim from

Seed (1979). 'Thus to prevent a dam being disrupted by a fault movement in the foundation may simply require the identification of potentially active faults and the selection of a site where such faults do not exist. Similarly the potential for settlement, slumping or tectonic movements, all of which could lead to loss of freeboard, can be ameliorated by the provision of additional freeboard so that the loss of some portion would not have serious consequences. In short, many of the potentially harmful effects of earthquakes on earth and rockfill dams can be eliminated by adopting defensive measures which render the effects non harmful. A list of such defensive measures would include the following:

– allow ample freeboard to allow for settlement, slumping or fault movements
– use wide transition zones (filters) of material not vulnerable to cracking
– provide ample drainage zones to allow for possible flow of water through cracks
– use a well graded filter zone upstream of the core to serve as a crack stopper
– provide crest details which will prevent erosion in the event of overtopping
– flare the embankment core at abutment contacts
– locate the core to minimize the degree of saturation of materials
– stabilize slopes around the reservoir rim to prevent slides into the reservoir
– provide special details if danger of fault movement in foundation exists.'

and 'Defensive measures, particularly the use of wide filters and transition zones, provide a major contribution to earthquake resistant design and should be the first consideration by the prudent engineer in arriving at a solution for problems posed by the possibility of earthquake effects.'

Sherard (1967) also discusses the matter of design of dams for earthquake and suggests a similar philosophy to that of Seed, i.e. to take defensive action, rather than attempting to rely too much on assessments of earthquake loadings and stresses. As well as some of the items listed above, Sherard includes locating a dam at a site with a rock foundation, rather than a site with a soil foundation; use of well graded sand/gravel/fines, or highly plastic clay rather than clay of low plasticity for the core (if the option is available). Other good practice would include using a narrow spillway, with high flood surcharge rather than a wide spillway with small surcharge so as to increase freeboard at full supply level; use of full width filters (i.e. 2.5 to 3 m, rather than using narrower zones constructed using spreader boxes); avoiding the use of a wave wall in concrete face rockfill dams and densifying loose-medium dense sands in the foundation rather than relying too heavily on results of dynamic analysis.

Clearly one will avoid use of dam construction methods which lead to low (relative) density saturated zones of sand or silty sand. Hence, hydraulic fill dams, and tailings dams constructed by the upstream method are unlikely to be satisfactory in areas subject even to moderate earthquake, and all filter zones and sandy gravel zones should be well compacted.

15.6.2 *Measures to improve liquefaction resistance*

USNRC (1985) discuss methods for altering the hazard, reducing the risk of liquefaction or improvement of stability during and after earthquake. These may apply to an existing dam or to new dam construction. They suggest there are four general classes of mitigation measures:

1. Changing operational procedures for the project, e.g.

– Lowering the maximum water level in the reservoir.
– Limiting public access to the area downstream of the dam.
– Institute early warning systems downstream.

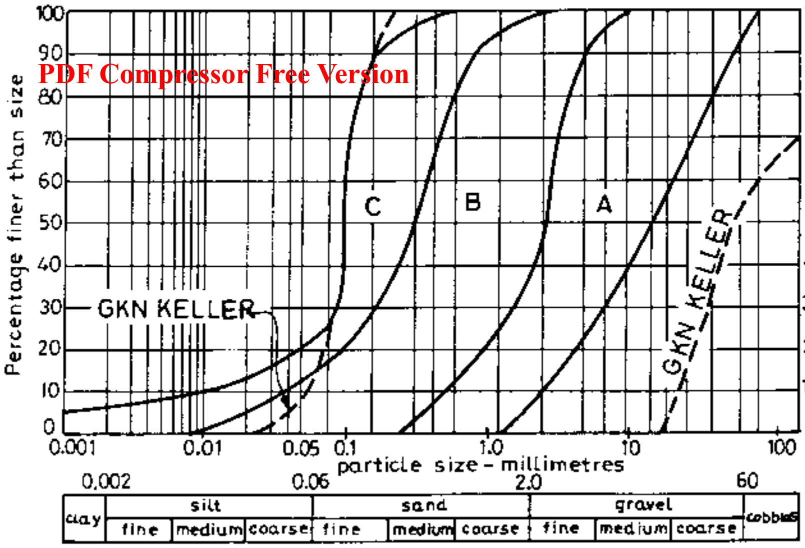


Figure 15.34. Soils suitable for vibroflotation (adapted from Brown 1977 and industry data).

2. Improve *in situ* foundation conditions to reduce liquefaction susceptibility including.

– Removal and replacement of unsatisfactory material, e.g. a) Excavation and engineered compaction of the existing soil, e.g. compaction of loose or medium dense sand. b) Excavation and engineered compaction of the existing soil with additives, e.g. cement stabilization. c) Excavation of existing soil and replacement with compacted non liquefiable soils, e.g. replacement of silt with gravel.

– Densification and increase in *in situ* lateral stress by *in situ* methods. a) Blasting. b) Compaction piles. c) Vibroflotation or vibratory probes. d) Compaction grouting. e) Dynamic compaction (consolidation).

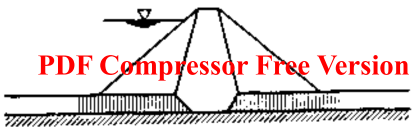
These methods are more likely to be applied to new construction than to remedial works for existing dams. Vibroflotation and dynamic consolidation are relatively common procedures, and particularly in the case of vibroflotation, high relative densities (70 to 85%) can be achieved at moderate cost to depth of up to 30 m in clean sands, sandy gravels and sands with minor silt. Brown (1977) discusses the applicability of the methods. Figure 15.34 shows the soils which can be treated by vibroflotation.

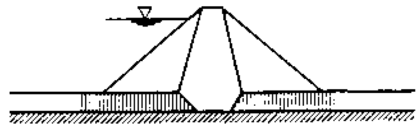
– *In situ* improvement by alteration of material, e.g: Mixing in-place material with additives e.g. lime, cement or asphalt introduced in augered piles; removing in-place material by jetting and replacing with suitable materials.

These methods are costly and unlikely to find wide application.

–Grouting or chemical stabilization. Chemical or cement grouts may be used to improve the strength and stiffness of soils. Figures 12.34 and 12.35. show the soils which can be treated in this way. Generally, grouting of soils is difficult and costly, and not likely to be the economic solution. It also has the disadvantage that permeability is reduced, lessening the ability of the soil to dissipate pore pressures built up by the cyclic loading of the earthquake.

Figures 15.35, 15.36 and 15.37 show schematically some of the techniques. USNRC (1985) gives additional details of the techniques and the applicability.





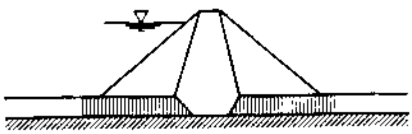


Figure 15.35. Schematic illustration of some in-situ improvement techniques to reduce the liquefaction hazard for earth dams (USNRC 1985, Silver 1985).

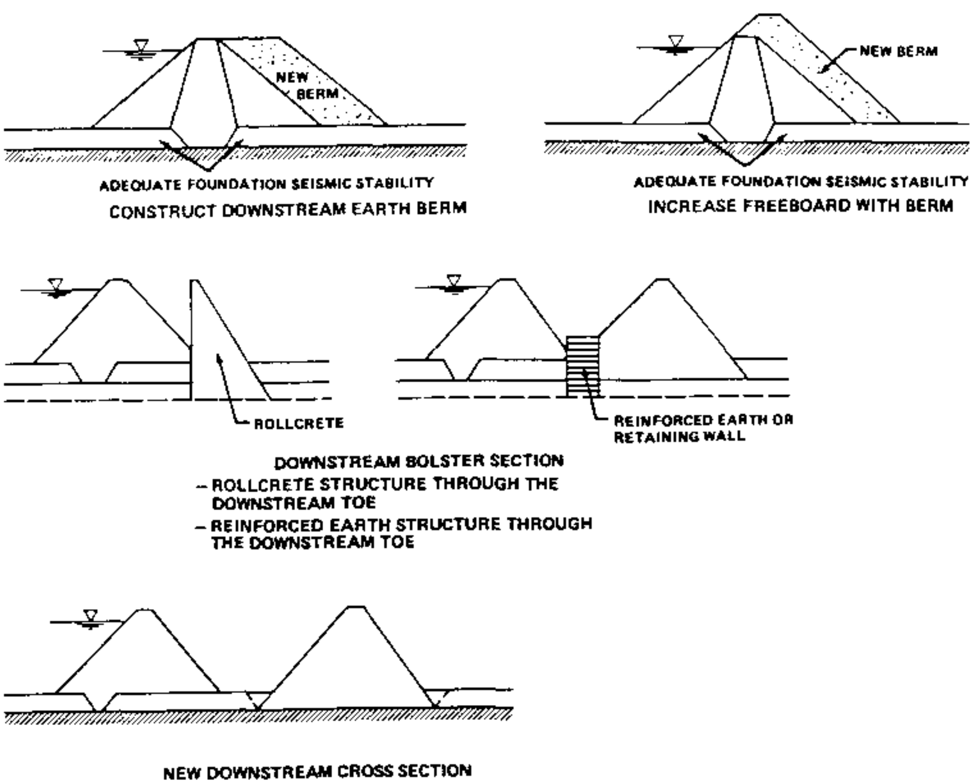


Figure 15.36. Schematic illustration of the use of structural measures to reduce the liquefaction hazard for earth dams (USNRC 1985, Silver 1985).

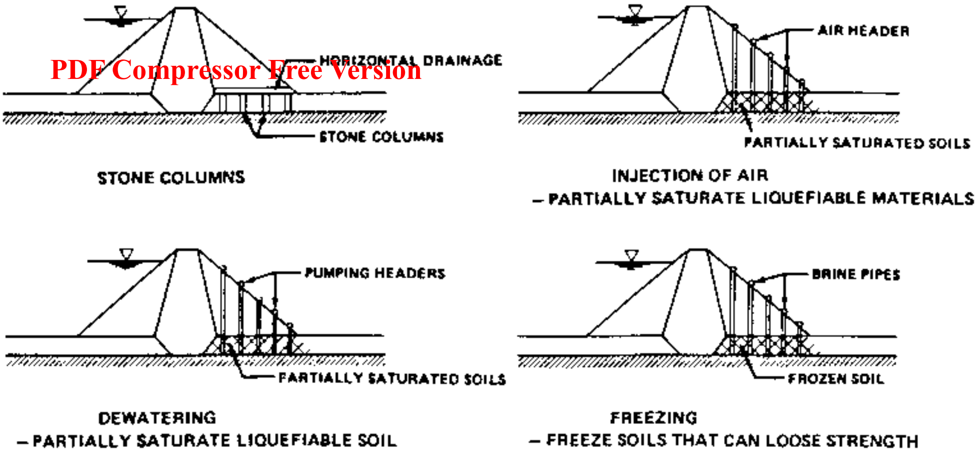


Figure 15.37. Schematic illustration of the use of groundwater control measures to reduce the liquefaction hazard for earth dams (USNRC 1985, Silver 1985).

3. Structural solutions.

– Berms, these may be added to an existing dam or be incorporated into a new dam design. They have several effects: a) The effective vertical stress on the foundation (of potentially liquifiable soil) is increased. This increases the cyclic shear strength and shear modulus. b) The static shear stresses are reduced, improving post earthquake stability. c) The freeboard may be increased by raising the crest level of an existing dam with a 'berm.'

– Piles, caisons, freezing etc. are more likely to be used for buildings than for dams because of the large costs involved.

– Rebuilding the structure – in extreme cases a new dam may be needed as shown in Figure 15.36.

4. Drainage solutions set out to relieve pore pressures built up during the cyclic loading of the earthquake by providing more permeable drainage paths, or by dewatering the soil to a partially saturated condition.

– Pressure relief wells. These may be conventional water wells with a stainless steel well screen, spaced at sufficiently close centres to allow dissipation of pore pressures from the cyclic loading. Since many alluvial soils are stratified, the horizontal permeability is much greater than the vertical, so the drainage wells will significantly improve the drainage. Stone columns may be used in lieu of conventional wells, and may be built by backfilling vibroflot probe holes with gravel. However, in this case control of erosion of fines into the stone columns may not be possible. In addition to helping dissipate pore pressures built up by the cyclic loading, pressure relief wells may be used to reduce uplift pore pressures under static conditions. Since 'liquefaction' is a process of reduction of effective stresses to zero by buildup of pore pressure, to start at lower static pore pressures may be critical. Figure 15.38 shows an example where pressure relief wells and a berm have been used to improve liquefaction resistance of an existing dam.

– Drainage layers. Drainage layers, i.e. horizontal drains, vertical drains, would be incorporated into the zones of the dam embankment to ensure that so far as practicable the embankment is partially saturated. If sands (e.g. tailings sand) is used to construct zones which will become saturated, layers of more permeable sand or gravel would be included to dissipate pore pressures.

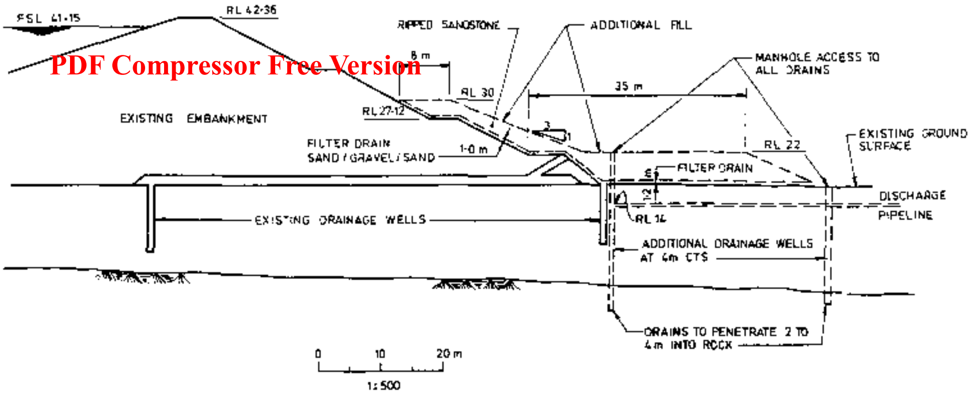


Figure 15.38. Schematic layout of pressure relief drains and berms to reduce liquefaction risk for Mardi Dam (PWD of NSW 1985).

– Dewatering and air injection. These are shown in principle in Figure 15.37 and are likely to be used as remedial measures in existing structures, and then only when other methods are not adequate. The major problem is that pumping or injection of air must be continuous.

As discussed in Section 15.5.1, it may well be good engineering practice to incorporate into the design one or more of these measures to improve the resistance of a dam foundation to liquefaction, rather than to rely on the ability to predict what will happen in a design earthquake. Positive measures to densify loose-medium dense sands, and/or to add a berm to reduce static shear stresses can be carried out at a relatively small incremental cost to a project, and improve the degree of confidence of the safety of the dam very significantly. In particular the risk of flow failure can be considerably reduced.

CHAPTER 16

Concrete face rockfill dams

16.1 GENERAL ARRANGEMENT AND REASONS FOR SELECTING THIS TYPE OF DAM

16.1.1 *Historic development of concrete face rockfill dams*

The development of concrete face rockfill dams (CFRD) is described by ICOLD (1989a) and Cooke (1984).

The first rockfill dam to have a concrete face was constructed in California in 1895. This followed on construction of timber faced dumped rockfill dams beginning in the 1850's. These often had steep (0.54H:1V to 0.75H:1V) slopes, with a skin of hand placed rocks to stop the face from ravelling. Then dams of greater height, up to 150 m high, were constructed with concrete faces and rockfill dumped in thick layers – often greater than 20 or 35 m, and placed without compaction other than sluicing.

The design of faced rockfill dams was (and is) mainly empirical, based on experience and judgement. The typical features of designs of CFRD up until the late 1950's are shown in Figure 16.1.

Many of these dams performed well. However several higher dams leaked excessively due to deformation of the concrete face, with resulting opening of joints and cracking. This could be attributed to the low modulus of the dumped rockfill, and to the detailing of joints, which allowed compression of horizontal and vertical joint fillers in the central part of the dam face (which is under compression) and resulting increased opening of other joints, including the perimetral joint. The leakage did not endanger the dam stability but in some cases was unacceptably high for operating reasons.

Over the period 1955 to 1965 there was a general adoption of compaction of rockfill. This was brought about by a realisation that dumping and sluicing of rockfill led to significant segregation, with the accumulation of larger rock at the base of the layer leaving large voids, and being particularly compressible. There was also a realisation that weaker rocks tended to lose strength on saturation leading to settlement if placed as dumped rockfill. Cooke (1984) and ICOLD (1989b) attributes the major change in approach to Terzaghi (1960a). Terzaghi (1960b) also introduced the change from a deep cutoff trench in rock as shown in Figure 16.1, to the adoption of the use of a toe slab, or plinth, on the grounds that excavation for the trench could loosen and fracture the rock, making it more permeable, and that the toe slab could be an adequate cutoff if founded on suitable rock, grouted and anchored to the rock with steel bars.

These developments, along with other refinements in design, have resulted in the earlier

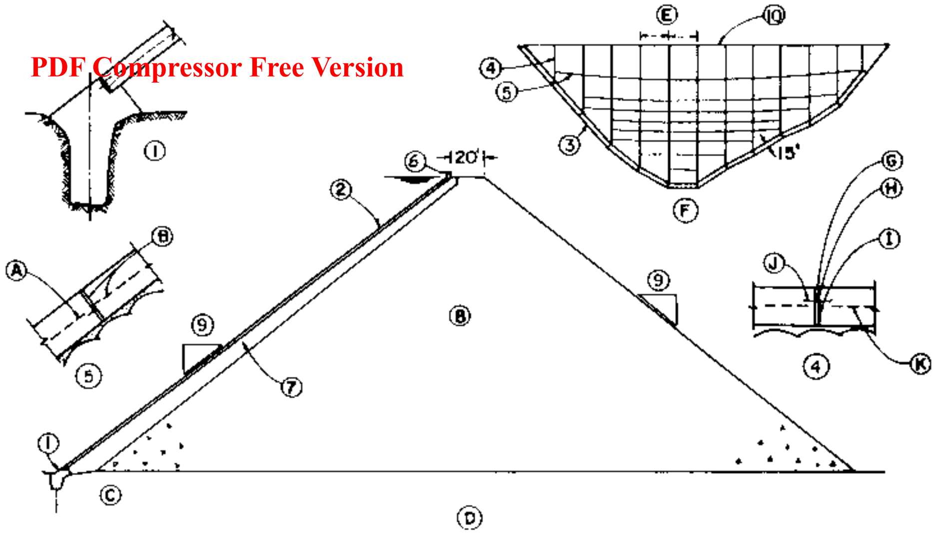


Figure 16.1. Features of early concrete face rockfill dam design (ICOLD 1989a). (1) Cutoff trench, (2) Concrete face, (3) Toe slab, (4) Vertical joint, (5) Horizontal joint, (6) Parapet, (7) Crane-placed large rock, (8) Dumped rockfill, (9) Slope, (10), Curved axis. (A) Reinforcement, (B) 1.9 cm redwood filler and Z waterstop, (C) Grout curtain, (D) Cross section of dam, (E) 18 m (60 feet), (F) Elevation of face, (G) Mastic, (H) Premolded asphalt, (I) Compressible joint filler, (J) U copper, (K) Reinforcement.

problems being overcome and acceptance of CFRD for the construction of many dams, including dams up to 180 m high. CFRD up to 220 m high are in the design phase.

16.1.2 *General arrangement – Current practice*

Figure 16.2 shows the current practice for zoning of CFRD constructed of sound, free draining rockfill, on a strong rock foundation.

The dam consists of:

Toe slab. A reinforced concrete slab cast on sound, low permeability rock to join the face slab to the foundation.

Face slab. Reinforced concrete, preferably between 0.25 and 0.6 m thick, with vertical, some horizontal, and perimetric joints to accommodate deformation which occurs during construction and when the water load is applied.

Zone 2D. Transition rockfill, processed rockfill, grading from silt to cobble size. The transition provides uniform support for the face slab, and acts as semi-impervious layer to restrict flow through the dam in the event that cracking of the faceplate or opening of joints occurs.

Zone 2E. Fine rockfill, selected fine rock which acts as a filter transition between Zone 2D and Zone 3A in the event of leakage through the dam.

Zone 3A . Rockfill, quarry run free draining rockfill placed in layers about 1 m thick. This zone provides the main support for the face slab and is compacted to a high modulus to limit settlement of the face slab.

Zone 3B . Coarse rockfill, quarry run free draining rockfill placed in layers about 1.5 to 2.0 m thick. Larger rock may be pushed to the downstream face. This zone is less affected by the

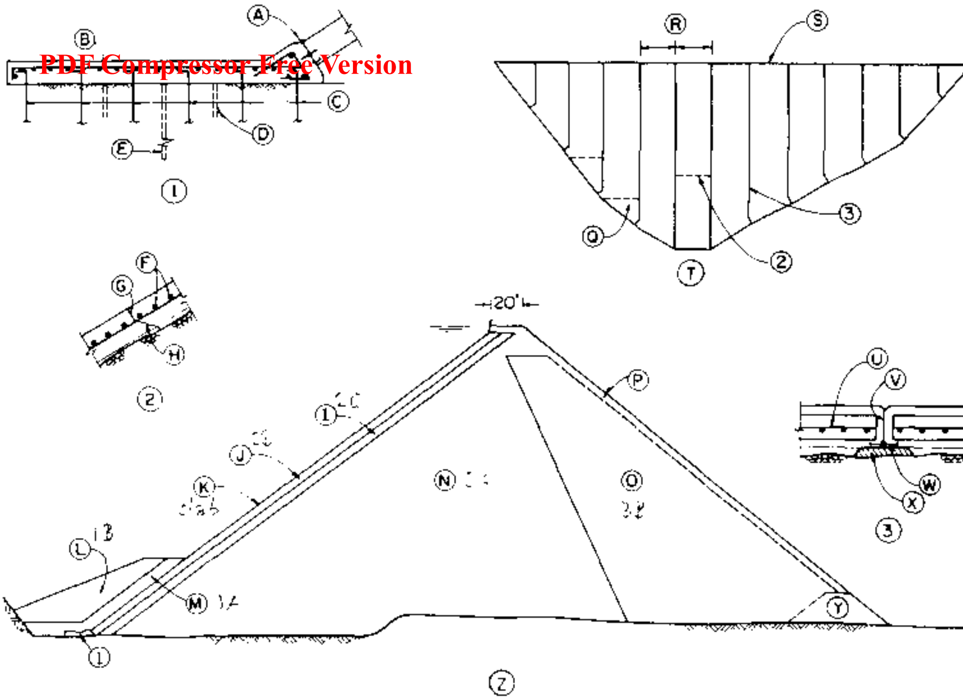


Figure 16.2. Current practice for zoning of CFRD constructed of sound rockfill on a strong rock foundation (adapted from ICOLD 1989a). (1) Toe slab, (2) Horizontal joint, (3) Vertical joint. (A) Perimetric joint, (B) Steel reinforcement, (C) Anchor bars, (D) Consolidation grout holes, (E) Grout curtain, (F) Horizontal reinforcement, (G) With form, (H) Broom joint, (I) Zone 2E, selected small rock placed in the same layer thickness as Zone 2D, (J) Zone 2D, processed small rock, (K) Concrete face, (L) Zone 1B, random, (M) Zone 1A, impervious soil, (N) Zone 3A, quarry run rockfill or gravel fill, about 1.0 m layers, (0) Zone 3B, quarry run rockfill or gravel fill, about 1.5 to 2.0 layers, (P) Available large size rock dozed to face, (Q) Starter slab, (R) 18 m (60 feet), (S) Straight axis, (T) Elevation of face, (U) Horizontal reinforcement, (V) Surface painted with asphalt, (W) Copper waterstop, (X) Mortar pad, (Y) Zone 3D, plus 0.3 m rockfill, (Z) Section of dam.

water load than Zone 3B, so a lower modulus is acceptable. The thicker layers allow placement of larger rock.

Many dams include a concrete crest wall which reduces the quantity of rockfill required.

Many variations of this zoning are adopted to meet site conditions and the quality of construction materials available. As discussed in Section 16.5.1, non free draining rockfill may be used, provided free draining zones are incorporated. In some dams, low permeability earthfill is placed upstream of the face (Zones 1A and 1B in Fig. 16.2) to control leakage in the event of it leaking. This is discussed in Section 16.5.4. The zone designation discussed above is used throughout the book. The transition zones 2D and 2E have been so designated in order to differentiate from Zones 2A, 2B and 2C used in earth and rockfill dams. There is no consistent terminology in use throughout the world.

16.1.3 *Site suitability and advantages of concrete face rockfill dams*

CFRD are suited to dam sites with a rock foundation and a source of suitable rockfill. In many cases CFRD will be a lower cost alternative than an earth and rockfill dam. This is discussed in Sherard & Cooke (1987), Fitzpatrick et al. (1985). Factors which may lead to CFRD being the most economic alternative include:

– the non availability of suitable earth fill;

– climate. CFRD are suited to wet climates, which may give short periods in which earthfill can be placed. This can result in significant overall savings in schedule;

– grouting for CRFD can be carried out independently of embankment construction, which may result in savings in overall time for construction;

– total embankment fill quantities are likely to be smaller and side slopes steeper for CFRD than for earth and rockfill dams leading to reductions in the cost of fill and diversion tunnels. Sherard & Cooke (1987) indicate that the cost of the concrete face is often less than the additional costs of earthfill and filters, and more extensive foundation treatment for earth and rockfill dams. This has commonly been the case in Australia where the majority of recent major dams have been CFRD.

Sherard & Cooke (1987) point out that CFRD have generally been used for dams of moderate height (40 m) or higher. They suggest that CFRD may also be economic for lower dams if they have a long crest length because:

– the cost of foundation treatment for a long, low dam is high relative to the overall cost, but will be less for CFRD because the CFRD requires a smaller width to be treated than for earth and rockfill

– the cost of filters in a long, low earth and rockfill dam is relatively high because the filters constitute a high percentage of the total volume.

They describe the use of CFRD for a 2000 m long and 20 m high dam in Venzezuala.

Varty et al. (1985) indicate that the Hydro-Electric Commission of Tasmania are using CFRD for several smaller dams, ranging in height from 20 to 44 m. They list faceplate construction techniques which may be changed from their normal practice, for these smaller dams, including the elimination of starter bays, a lighter slip-form and a mobile crane on the dam crest.

These examples show that smaller dams can be economically constructed as CFRD, but probably where the economy of scale applies, either as a long crest, or in a requirement to construct several dams, to offset the cost of setting up for slip-forming.

Sherard & Cooke (1987) argue that CFRD should be considered for the very highest dams. They point out that at least 6 CFRD's with heights 180 to 220 m were in final design stages in 1987. They argue that CFRD have fundamental advantage over earth and rockfill dams in that there is no possibility of piping erosion of the earth core and over arch dams in that they rely on gravity for stability, not high strength abutments, and that the CFRD supports high abutments, rather than stressing them. They argue that there is no reason why CFRD's up to 300 m high could not be built and suggest that the jump in precedent from current heights is not excessive, and is similar to practice in earth and rockfill dams. They suggest that conservative perimetric joint details would have to be used for very high CFRD's, and wider, more conservatively designed processed material used for Zones 2D and 2E to ensure homogeneity, lack of segregation and low permeability.

Fitzpatrick et al. (1985) suggest that because of the high modulus obtained with compacted gravel fills, they are preferable construction material for very high dams. They counsel caution in the use of rockfill for dam heights greater than have so far been built.

16.2 ROCKFILL ZONES AND THEIR PROPERTIES

16.2.1 *Zone 2D – Transition rockfill*

In CFRD's constructed in the 1960's the function of Zone 2D was seen to be to provide uniform support for the face slab. Specifications required that the Zone 2D was screened to remove all materials less than 25 or 50 mm. This was to ensure that, in the event of a leak in the face slab, there would be no fines to be washed away which might lead to loss of support of the face slab. However this also meant that Zone 2D was very permeable, rocks were easily dislodged during construction, and a rough, porous surface led to excess concrete being required for the face slab.

Beginning with the 110 m high Cethana Dam in 1971 (Wilkins et al. 1973) there was a change in design approach which resulted in Zone 2D being specified as 'crusher run' or 'quarry run' (after passing through a grizzly) rockfill passing 150 to 225 mm (up to 300 mm). Table 16.1 gives some Zone 2D gradation specifications.

This resulted in a material with a lower permeability, and a smoother and more stable surface on which to construct the face slab. Many dams have been successfully constructed in this way and this was the established practice up till about the mid 1980's.

Sherard (1985) points out that quite large leakage had occurred at some dams constructed with crusher-run Zone 2D including 1800 litres/second at the Alto Anchicaya Dam on first filling. Sherard notes that, for such wide graded materials, with a small percentage of sand size material, segregation was a problem, and this could in part explain why such large leakage had occurred. He suggests that about 40% of sand size (passing 4.76 mm) is required to avoid the segregation. Sherard also shows that leakage through the dam resulting from a crack in the face slab, is controlled more by the permeability of Zone 2D, than by the crack aperture. Sherard concludes that Zone 2D should be a 4 to 5 m wide zone with the gradation in Table 16.2 being desirable.

Sherard (1985) indicates that such a grading will be 'stable,' i.e. not susceptible to washing out of the fines, and that a permeability of 10^{-6} m/sec would be achieved by such a grading for Zone 2D. This is confirmed by Keming (1987).

ICOLD (1989a) recommend a Zone 2D grading as shown in Table 16.2, i.e. virtually the same as that suggested by Sherard. They indicate that a maximum of 10 to 12%, passing 0.075 mm is desirable, while giving 15% as the upper limit. ICOLD (1989a) indicates that Zone 2D should be 4 to 5 m wide, possibly wider for dams higher than 150 m.

Table 16.1. Zone 2D gradation specification for 3 dams, 1971-1982 (adapted from Sherard 1985).

Size	Allowable percent fines		
	Cethana	Alto Anchicaya	Foz do Areia
300 mm		100	
225 mm	100		
150 mm	80-100		100
75 mm	51-100	50-100	55-100
19 mm	15-63	25-60	18-65
4.76 mm	0-40	0-30	5-22
0.42 mm	0-17	0-5	0-8
0.075 mm	0-5		0-1

Table 16.2. Desirable specification for Zone 2D.

Size	Sherard (1985) % finer	ICOLD (1989a) % finer
75 mm	90-100	90-100
37 mm	70-95	70-100
19 mm	55-80	55-80
4.76 mm	35-55	35-55
0.6 mm	8-30	8-30
0.075 mm	2-12	5-15

Sherard (1985) acknowledges that the required Zone 2D grading may require processing and/or blending of materials, but points out that the incremental cost is small.

Fitzpatrick et al. (1985) indicate that a Zone 2D width of 1m has been adopted in recent Tasmanian Hydro-Electric Commission (HEC) dams (reduced from 3 m because of the cost), with an increase of width of Zone 2E from 3 to 5 m to maintain the same overall width.

Zone 2D is usually compacted in 0.4 or 0.5 m thick layers with 4 to 8 passes of a 10 tonne vibratory roller. The face is also rolled as described in Section 16.4.1.

Fitzpatrick et al. (1985) and Sherard (1985) point out that an added advantage of having Zone 2D graded as shown in Table 16.2, is that, if a crack forms in the face slab, it can be sealed by spreading silty sand over the crack and having the silty sand wash into it, with further erosion controlled by Zone 2D acting as a filter. This method has been used to reduce leakage on several dams including Shiroro Dam in Nigeria (Bodtman & Wyatt 1985) and Khao Laem Dam in Thailand (Watakeekul et al. 1985).

16.2.2 *Zones 2E, 3A and 3B – Fine rockfill, rockfill and coarse rockfill*

The basic requirements for rockfill in a CFRD are:

– The rockfill should be free draining to avoid buildup of pore pressure during construction, and to allow drainage of water which might leak through the faceplate.

– The rockfill should have a high enough modulus after compaction in the dam to limit face slab deflections under water load to acceptable values. Creep of the rockfill should also be small enough to avoid excessive longer term settlements.

– It should be readily available as a quarry run product with a minimum of wastage of oversize or undersize rock.

An additional requirement is that: Zone 3A should be graded to meet filter criteria between Zone 2D and Zone 3A, and between Zones 3A and 3B, to avoid washing of fines from Zone 2D in the event of leakage through the face slab.

A wide range of rock types have produced satisfactory rockfill including granite, basalt, dolerite, quartzite, rhyolite, hornfels, limestone, gneiss, greywacke, andesite, welded tuff, and diorite. Rocks such as sandstone, siltstone, argillite, schist and shale have been used but in some cases produce a non-free draining rockfill. This is discussed further in Section 16.5.1. Gravels have also been used with success, and as discussed below, can lead to very high modulus fills. Cooke (1984) suggests that 'if blasted rockfill is strong enough to support construction trucks and the 10 tonne vibratory roller when wetted, it may be considered to be suitable for use in compacted rockfill.' He goes on to point out the need for drainage zones if the resulting rockfill is of low permeability.

Penman (1982) and Penman & Charles (1976) suggest that for rockfill to be considered 'free draining,' it should have a permeability of at least 10^{-5} m/sec based on *in situ* tests in the rockfill.

This is based on the requirement for adequate permeability to dissipate construction pore pressures in the rockfill, rather than having a high water discharge capacity from a leaking face slab.

As discussed by Cooke (1984) and Sherard & Cooke (1987), rockfill placed in the normal way, i.e. dumped from a truck, and spread by a bulldozer, will result in segregation, with the coarser particles collecting at the base of the layer, and the finer rock and fines on the surface. Breakdown of the upper part of the layer during rolling creates even more stratification. Cooke (1984) and Cooke & Sherard (1987) point out that far from being a problem, this stratification is desirable because:

– tyre wear on trucks is reduced and the smoother surface allows more rapid truck travel

– the smooth surface facilitates the rolling operation, spreading the vibrating load and reducing roller maintenance compared to an irregular rocky surface

– the lower parts of layers have a high horizontal permeability, facilitating drainage of leakage or in the event of embankment overtopping during construction. The resulting average horizontal permeability is much higher than if the fines were distributed uniformly throughout a layer

– the layers create a variation in vertical permeability which will prevent buildup of pore pressures in the rockfill.

It should be noted that this stratification means that 'free draining rockfill' may allow water to pond on the surface of layers, provided the lower parts have higher permeability.

As pointed out by Cooke (1984), there is no need to scarify the surface of layers of compacted rockfill prior to placing the next layer.

The discussion above largely relates to Zones 3A and 3B. Zone 2E is required to act as a filter to Zone 2D, and is therefore placed in relatively thin layers (0.4 to 0.5 m, the same as Zone 2D). This also ensures a high modulus. Zones 3A and 3B are placed in layers of the order of 1m, and 1.5 to 2 m thick respectively, which results in a gradation of permeability and a lower modulus for Zone 3B, which is acceptable since the water load is largely taken by Zone 3A. Rolling is usually by 4 (up to 8) passes of a 10 tonne vibratory steel drum roller.

Zone 2E is obtained by selecting suitable material in the quarry, or by passing rockfill over a grizzly. The filter requirement (compared to Zones 2D and 3A) is usually readily achieved by this process, and further treatment is not required.

The rockfill grading in Zone 3A (and to a lesser extent in Zone 3B) is important. A well graded rockfill will compact to a higher modulus than a poorly graded fill, and the amount of fines needs to be limited if the rockfill is to be truly free draining.

Cooke (1984) suggests that the grading specification should be:

– maximum size shall be that which can be incorporated in the layer and provides a relatively smooth surface for compaction

– not more than 50% shall pass 25 mm sieve,

– not more than 6% shall be clay size particles (this is taken to mean silt and clay).

In Sherard & Cooke (1987), it is suggested that it is better to specify:

– not more than 20% finer than 4.76 mm,

– not more than 10% finer than 0.075 mm (i.e. silt and clay).

They suggest that if the rockfill has a higher proportion of fines, 'the final evaluation of suitability can be made on the trafficability of the rockfill surface when the material is thoroughly wetted. A stable construction surface under travel of heavy trucks demonstrates that the wheel loads are being carried by a rockfill skeleton. An unstable construction surface, with springing, rutting, and difficult truck travel, shows that the volume of soil like fines is sufficient to make the rockfill relatively impervious. Where the surface is unstable, the fines dominate the behaviour and the resulting embankment may not have the properties desired for a pervious rockfill zone.'

Cooke & Sherard (1987) point out that there is no technical need for rock in the rockfill to have a high compressive strength – rockfills constructed of rocks with compressive strength of 30 to 40 MPa are no more compressible than those of higher strength. Rocks with very high compressive strength lead to higher quarrying cost and wear on equipment. They conclude that any rock with a (soaked) unconfined compressive strength of 30 MPa or more is adequate. Lower strength rocks may be used with special zoning provisions as discussed in Section 16.5.1.

Penman (1982) discusses the need for well graded material for rockfill, showing that the contact stresses for single sized large rock is very high, and will result in overstress on the contact points giving settlement and creep effects.

Cooke (1984) discusses the use of gravel as compacted rockfill and points out that, if available, this material can make very suitable rockfill. He points out that:

– gravel is often more economically handled than rockfill (lower excavation and loading cost, and less wear on tyres and rollers);

– compacted gravels commonly have high modulus of compressibility, 5 to 10 times that of compacted rockfill. Since face slab movements vary roughly inversely with the modulus, and directly with the square of height of the dam, gravel fills are desirable for higher dams;

– if a significant fines content is allowed, then chimney drains, abutment drains, filters and intermediate drainage layers may be required.

Amaya & Marulanda (1985) describe the use of gravel for the 125 m Golillas Dam in Colombia. In this dam the bulk of the fill (i.e. Zone 2 in Fig. 16.3) was natural river gravels. The grading is shown in Table 16.3.

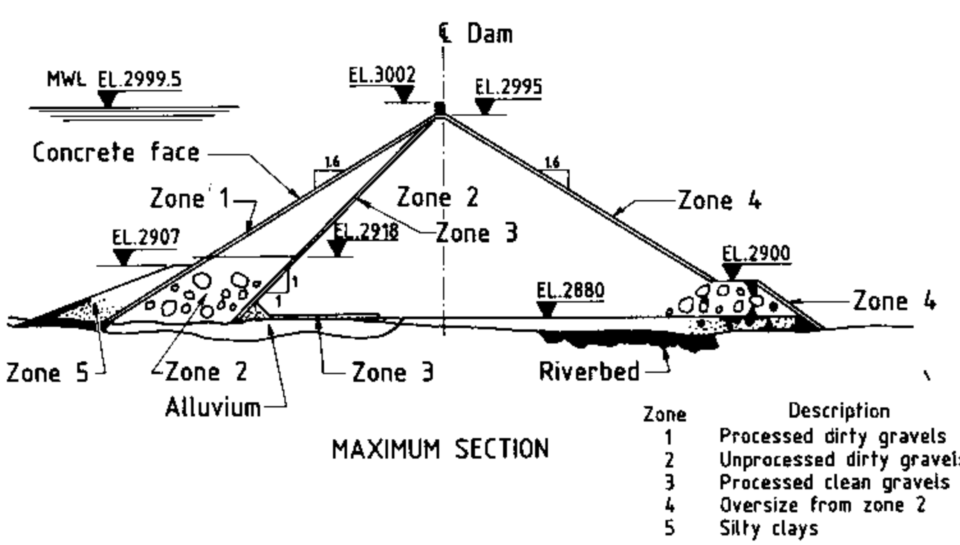
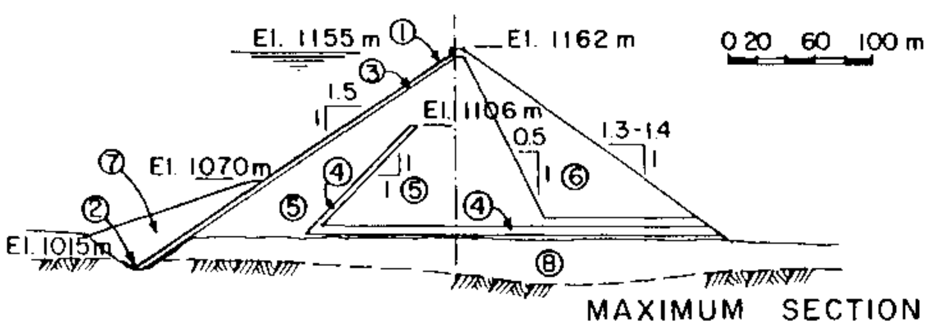


Figure 16.3. Golillas Dam (Amaya & Marulanda 1985).

Table 16.3. Grading of river gravels used as 'rockfill' in CFRD.

Sieve size	Dam Golillas	Salvajina % passing	Crotty
300 mm	100	100	100
150 mm	60-100	65-100 (98) (b)	85-100 (98)
75 mm	35-80	35-100 (80)	34-98 (85)
25 mm	20-50	10-80 (40)	26-70 (55)
4.76	10-30	0-30 (8)	6-42 (16)
1.18	4-22	0-25 (6)	2-22 (10)
0.075	0-12(a)	0-12 (3)	0-4 (2)

Notes: (a) Average 8% for one source, 12% for second; (b) Average.



ZONING	MAXIMUM SIZE in - (cm)	THICKNESS LAYER in - (cm)
① Face slab	–	–
② Toe slab	–	–
③ Zone 1 - Compacted semipervious	4 (10)	18 (45)
④ Zone 2A - Drain	16 (41)	24 (60)
⑤ Zone 2 - Compacted gravels	12 (30)	24 (60)
⑥ Zone 4 - Compacted rockfill	24 (60)	36 (90)
⑦ Zone 5 - Impervious	1-12(25-30)	12 (30)
⑧ Alluvium	–	–

Figure 16.4. Salvajina Dam (Sierra et al. 1985).

The Golillas Dam gravels were made up of 70% sandstone, 20% siliceous shales and 10% siltstones and limestones. The gravels were washed for drainage zones in the dam. Sierra, Ramirez & Hacelas (1985) describe the use of gravels in Salvajina Dam in Colombia. The 148 m high dam had its 'Zone 3A' constructed of gravels from gold mining dredging for Zone 2 as designated in Figure 16.4. The range and average grading is given in Table 16.3. The Hydro-Electric Commission of Tasmania used gravels to construct the 82 m high Crotty Dam. Gradings are also given in Table 16.3.

Figures 16.3, 16.4 and 16.5 show the cross sections of these dams. Note that where gravel is used for the whole of the equivalent of Zones 3A and 3B, flatter slopes are required.

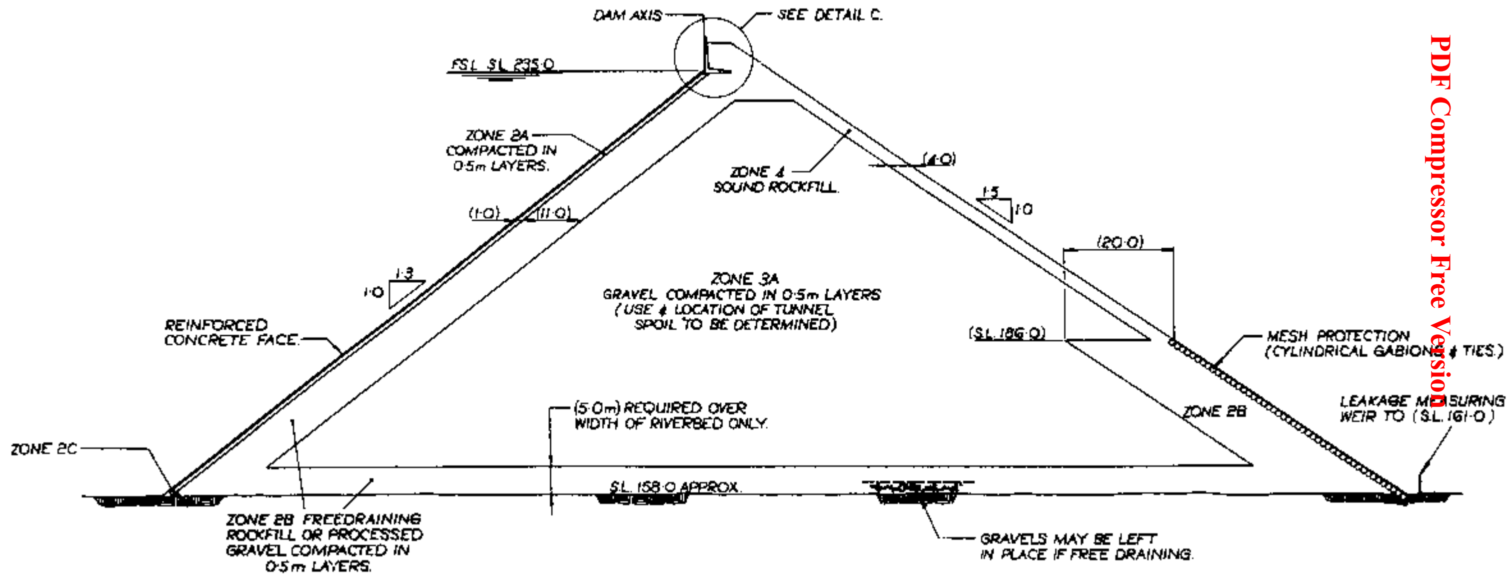


Figure 16.5. Crotty Dam (HEC 1988).

Table 16.4. Properties of compacted rockfill.

	Alto Anchicaya	Bastyan	Cethana	Foz do Areia(1)	Khao Laem	Little Para	Lower Pieman	Mackintosh
Dam dimensions								
Height (m)	140	75	110	160	130	53	122	75
Crest length/height	2	5	2	5	7	5	3	6
Rock used in rockfill								
Geology	Hornfels	Rhyolite	Chert	Basalt/Breccia	Limestone	Schist	Quartzite, slate, dolomitic siltstone	Greywacke
Unconfined strength (MPa)	–	–	–	235/37	190	–	–	–
Soaked/unsoaked strength	–	–	–	0.8	1.0	–	–	–
Uniformity coefficient	38	–	–	6	–	200	–	–

Rockfill placement and compaction								
Layer thickness (m)	0.6	1.0	0.9	0.8 (3A) 1.6 (3B)	1.0 (3A) 2.0 (3B)	1.0	1.0	1.0
Roller mass (tonnes)	10	10	10	10	10	10	10	10
Number of passes	4	8	8	4	4	4	8	8
Void ratio achieved	0.3	–	–	0.33	–	0.27	–	–
% water added by volume	20	–	–	25	15	15	–	–
Rockfill modulus (MPa)								
During construction	100 to 170 138 average	160	145	35-55 (3A) 25-35 (3B)	50 (3A) 40 (3B)	38	160	40
First filling		300	310	90	165		–	95

	Mangrove Creek (2)	Murchison	Paloona	Salvajina	Serpentine	Shiroro	Tullabardine	Wilmot	Winneke
Dam dimensions									
Height (m)	80	94	38	148	39	125	26	35	85
Crest length/height	6	2	4	2.5	3	4	8	4	6
Rock used in rockfill									
Geology	Sandstone and siltstone	Rhyolite	Chert	Sandy gravels	Quartzite	granite	Greywacke	Greywacke and quartzite	Siltstone
Unconfined strength (MPa)	45 to 64	–	–	–	–	–	–	–	75 to 100
Soaked/unsoaked strength	0.5	–	–	–	–	–	–	–	–
Uniformity coefficient	300	–	–	8	–	38	–	–	50
Rockfill placement and compaction									
Layer thickness (m)	0.6 semi pervious 0.45 random	1.0	–	0.6	–	1.0	1.0	–	0.9 0.5 random
Roller mass (tonnes)	10	10	–	10	–	15 to 20	–	–	10
Number of passes	4	8	–	4	–	6	10	–	4, 6 random
Void ratio achieved	0.25	–	–	0.25	–	0.2	8	–	–
% water added by volume	5 to 7-½	20	–	None		15	–	–	15
Rockfill modulus (MPa)									
During construction	60 semi pervious 100 random	225	75	49 @ 0.24 MPa 393 @ 0.74 MPa	115	55 to 103 76 average	90	115	55
First filling	–	650	115	*	95		170	160	

Notes: (1) Foz do Areia Dam was 75% basalt, 25% Breccia. Zone 3A constructed in 0.8 m layers, Zone 3B in 1.6 m layers. (2) Mangrove Creek Dam has a semi impervious zone under the faceplate, and random rockfill forms the bulk of the fill.
*Twice construction values.

The gravels are placed in approximately 0.6 m thick layers and compacted with 4 passes of a 10 tonne steel drum vibratory roller, usually without water added. Water can make compaction of the silty gravels difficult.

Very high moduli can be obtained as shown in Table 16.4.

As pointed out by Cooke (1984) gravel has been successfully used in several high earth and rockfill dams including Nurek (305 m), Oroville (244 m), Mica (244 m) and Bennett (183 m). Moduli of 365 MPa for Oroville and 551 to 689 MPa for Bennett were achieved despite use of smaller rollers than would now be adopted.

16.2.3 *Effect of rock properties and compaction on modulus of rockfill*

As has been discussed above, the modulus of compressibility (E) of the rockfill is dependent on the rock type, strength, shape, gradation of rock sizes in the rockfill and layer thickness. It is also dependent on the roller size and type, number of passes, whether water is added during compaction, the confining stresses on the rockfill, and also the duration of loading, i.e. there is a creep component.

Table 16.4 summarizes the properties of compacted rockfill from a number of CFRD's. The construction and first filling values of rockfill moduli have been calculated from observed settlements of rockfill during construction of the dam, and then from the observed deflection of the face slab on first filling. Unless otherwise stated, the values quoted are generally 'average,' typical of rockfill at the centre of the dam. The moduli for the Hydro-Electric Commission dams (Wilmot and Lower Pieman) have been calculated with the simplified procedure shown in Figure 16.6. Fitzpatrick et al. (1985) indicate that this gives values which are similar to more rigorous finite element methods.

The following points can be made:

– The use of high strength rock does not guarantee a high modulus, e.g. Foz do Areia Dam. The relatively low modulus for this dam is due to poor grading, i.e. a lack of sand size particles (Pinto et al. 1985), and is typical of basaltic rocks in that region. The void ratio of the compacted rockfill is a guide to their behaviour (note that Foz do Areia Dam has a relatively high void ratio of 0.33. Most rockfills have a void ratio less than 0.25).

– Low strength rocks, which break down significantly, e.g. Kangaroo Creek, Little Para and Mangrove Creek, can give quite high moduli, but it must be remembered that they will probably also give low permeability.

– The highest moduli are achieved for gravels, where the rounded shape limits crushing of

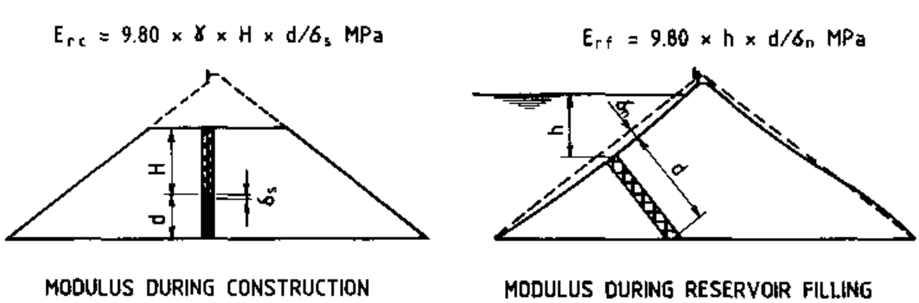


Figure 16.6. Method of calculating rockfill moduli (Fitzpatrick et al. 1985). E_{rc} = modulus during construction; E_{rf} = modulus during reservoir filling.

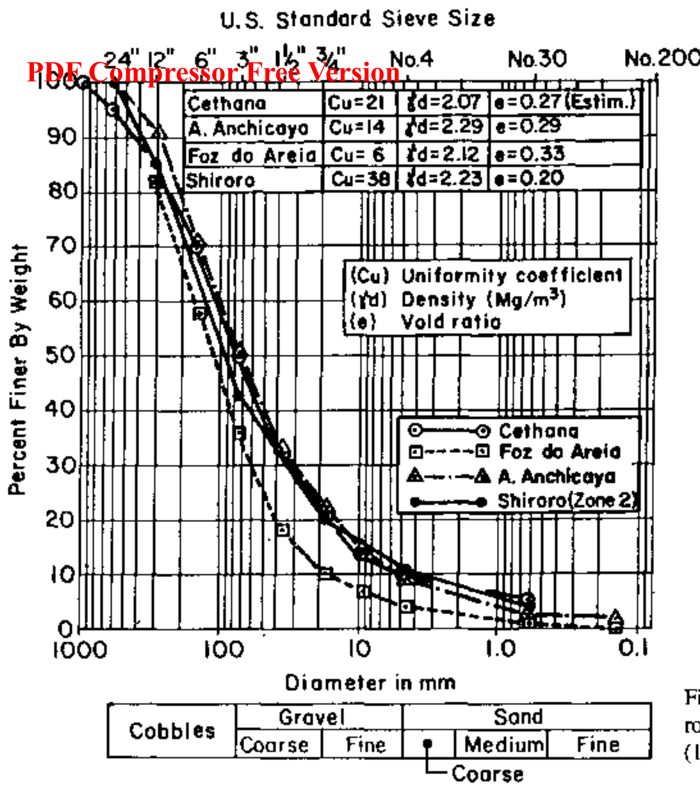


Figure 16.7. Properties of dam rockfills, Bodtman & Wyatt (1985).

Table 16.5. Modulus of compressibility at selected concrete face rockfill dams (adapted from Bodtman & Wyatt 1985).

Dam	Rock type	Modulus of compressibility
Cethana	Quartzite	138
Alto Anchicaya	Hornfels	138
Shiroro	Granite	76
Wilmot	Greywacke	69
Paloona	Argillaceous chert	55
Foz do Areia	Basalt	34

the contact points between particles of the fill. Conversely the high stresses on the contact points in the poorly graded basaltic fill cause the low modulus.

– Well graded high strength rockfill can also give high moduli, e.g. Murchison, Bastyan and Lower Pieman.

– Additional compaction (e.g. Murchison 8 passes) and thin layer (e.g. Alto Anchicaya) can lead to higher moduli

– The modulus is apparently affected by the valley shape. Arching stresses in narrow valleys provide additional confinement, which leads to higher moduli. Fitzpatrick et al. (1985) and Bodtman & Wyatt (1985) discuss this. Rockfill details are shown in Figure 16.7, and resulting moduli are given in Table 16.5. The latter point out that despite the gradation of Shiroro Dam rockfill being almost identical to that of Cethana and Alto Anchicaya, and Shiroro being compacted to a lower void ratio, the rockfill at Shiroro has a lower modulus, probably due to valley shape effects.

– The apparent modulus on first filling is commonly 2 to 3 times that observed during construction. This is explained by the fact that the modulus is dependent on the confining stress. This is important as the face slab displacement is inversely proportional to the 'first filling' modulus

– The modulus is dependent on layer thickness, e.g. at Foz do Areia and Khao Laem dams where Zone 3B, compacted in layers twice the thickness of Zone 3A had moduli 60 to 80% of that for Zone 3A.

It will be noted that in all dams water was added to the fill, usually at a rate of 15 to 25% of the rockfill volume. This results in reduced compressibility although the improvement may be marginal in some cases, as shown by roller trials for Boondooma Dam (see Section 14.4.2). In this case the rockfill was high strength rhyolite (100 MPa compressive strength).

Cooke & Sherard (1987) conclude that: '(1) For most hard rocks and CFRD of low to moderate height, the addition of water has negligible influence on the dam behaviour. (2) For high dams and for rocks which have significantly lower unconfined compression strengths when tested in saturated conditions (than when tested dry), water should probably be added routinely for the upstream shell (Zone 3A). (3) For rocks with questionably high contents of earth and sand size particles, water should nearly always be used. For dirty rock, the water softens the fines so that the larger rocks can be forced into contact with each other by the vibrating roller.'

As pointed out by Cooke & Sherard (1987), it is not intended that the application of water will wash fines into the voids, and hence the use of a high pressure nozzle is not necessary. Adding water in the truck prior to dumping on the surface is practicable and economical (Varty et al. 1985).

Some indication of likely rockfill behaviour will be obtained from consideration of rock type, unconfined compressive strength, and jointing in the rock in the quarry.

The best guide to rockfill behaviour in the investigation phase of a project will be to construct test fills, and carry out plate bearing tests on the compacted fill. This, coupled with measurement of grading and voids ratio of the resulting rockfill, and consideration of the height of the dam, shape of the valley and comparison with other projects, should enable a reasonable guide to expected rockfill modulus.

Laboratory tests can be used to estimate the modulus. These involve large diameter consolidation equipment. Bowling (1980) describes such tests and their relationship to observed behaviour. Penman (1982) and Charles (1976) also discuss the use of laboratory oedometer tests. Because of the large size of particles in rockfill, it is normal to use a 'model' rockfill. Charles (1976) discusses this, and concludes that reasonable predictions of modulus can be obtained by such tests.

16.2.4 *Selection of side slopes and analysis of slope stability*

When the CFRD is constructed of hard, free draining rockfill the upstream and downstream slopes are fixed at 1.3H to 1V or 1.4H to 1V, which corresponds roughly to the angle of repose of loose dumped rockfill, and prevents ravelling of the faces.

When gravel is used for the dam 'rockfill' zones, flatter slopes are needed to prevent ravelling of the face. Usually 1.5H:1V has been adopted in these cases although 1.6H:1V has been used.

When weak rockfill has been used flatter slopes have again been adopted, e.g. 1.5H:1V for Mangrove Creek Dam (MacKenzie & McDonald 1985). If foundation strengths dictate, flatter slopes may also be required, e.g. 2.2H to 1V was used for Winneke Dam (Casinader & Watt 1985).

Haul roads may be needed on downstream slopes, and in this event steeper slopes between the 'berm' located by the haul road may be used, e.g. 1.25H:1V was used for Foz do Areia Dam (Pinto et al. 1985).

In some cases, the upper 10 to 15 m of the dam may be steepened to as much as 1.25H to IV to provide the camber of the crest (Fitzpatrick et al. 1985).

As pointed out by Sherard & Cooke (1987b), and Fitzpatrick et al. (1985), the stability of the slopes in the dam are not usually analysed. This is in recognition of the fact that CFRD's have no pore pressures in the rockfill and will remain stable under static loads when constructed to the slopes described above. However, if analysis is to be carried out, a knowledge of the shear strength properties is required.

Rockfill does, in reality, have a curved Mohr envelope, yielding high friction angles at low confining stress, which is seldom taken into account when selecting effective friction angles for the rockfill. Penman (1982) quotes results on testing by Charles & Watts (1980) which shows that the shear strength relationship for rockfill at confining pressures less than 700 kPa is:

$$\tau = A(\sigma')^b$$

where A and b are constants for the rockfill and σ' and τ are measured in kN/m^2. Table 16.6 gives values for four rockfills, and resulting friction angles for a range of confining stresses.

Since much rock testing would be done at high confining stresses, the friction angle at low stresses could be underestimated.

Charles & Watts (1980) show that for an infinite slope at an angle Θ to be horizontal, built of rockfill with a compacted unit weight of γ and shear strength given by $\tau = A(\sigma')^b$, the factor of safety will be given by:

$$F = \frac{A \cos^{(2b-1)}\Theta}{\gamma^{(1-b)} \sin \Theta \, z^{(1-b)}}$$

where z is the depth of the slide surface and assuming no pore pressure. When this is applied to the slate rockfill properties in Table 16.6, $\gamma = 22$ kN/m^3 and a slope Θ of 45°, the factor of safety is 1.4 for an infinite slope of $z = 20$ m, and 1.65 for $z = 10$ m.

The factor of safety is higher for non infinite slopes, e.g. for a 50 m slope, z = 20 m, F = 1.8.

The stability of CFRD under earthquake loading is discussed in Chapter 15.

Table 16.6. Variation of friction angle of rockfill with confining stress (adapted from Charles & Watts 1980).

Rock type	Strength coefficients		Friction angle for confining stress		
	A	B	100 kN/m^2	300 kN/m^2	700 kN/m^2
Basalt	4.4	0.81	61°	56°	52°
Sandstone	6.8	0.67	56°	46°	38°
Soft slate	5.3	0.75	59°	52°	46°
Slate	3.0	0.77	46°	39°	34°

Note: Friction angle is secant angle assuming $c' = 0$.

16.3 CONCRETE FACE

16.3.1 *Toe slab*

The principal purpose of the toe slab (or 'plinth'), is to provide a 'watertight' connection between the face slab and the dam foundation. The toe slab is usually founded on strong, non erodible rock which is groutable, and which has been carefully excavated and cleaned up with a water jet to facilitate a low permeability cutoff. For these conditions the toe slab width is of the order of 1/20 to 1/25 of the water depth (ICOLD 1989b, Cooke & Sherard 1987). Up the dam abutment, the width is changed according to the water head. This is done in several steps, not gradually, for construction convenience. The minimum width has generally been 3 m, although Cooke & Sherard (1987) suggest that for dams less than 40 m high on very good rock, 2 m could be used. For poorer rock conditions a wider toe slab and/or other erosion control measures may be used. This is discussed in Section 16.5.2.

The minimum toe slab thickness is usually between 0.3 and 0.4 m, but may be up to 0.6 m for the lower toe slabs of high dams. The actual thickness is usually more because of the need to fill over-excavation, and to make up for irregularities in the topography. Where this extra concrete is significant it is common to construct the toe slab in two stages, the first stage is to fill the irregularities.

Figures 16.8 and 16.9 show toe slab designs for Mangrove Creek, Boondooma, Cethana and Lower Pieman dams. These designs are typical for CFRD's.

It is necessary to ensure that the toe slab is stable under the imposed forces. For a toe slab of normal thickness there is adequate friction resistance on the base, unless there are unfavourably oriented low strength bedding planes, joint or shears in the foundations. In any case, the toe slab is usually anchored to the rock with grouted dowells which are generally 25 to 35 mm diameter, reinforcing steel bars, 3 to 5 m long, and are installed at 1.0 to 1.5 m spacing. The bars are grouted full length into the rock and hooked on to the layer of reinforcing steel in the toe slab. The anchors are provided nominally to prevent uplift during grouting, although Cooke & Sherard (1987) claim uplift will not develop in most cases.

For toe slabs which are thicker than normal, due to overbreak or irregularities in the foundation, the stability of the toe slabs should be analysed assuming the uplift pressure under the slab is zero at the downstream toe and varies linearly to full reservoir head at the upstream toe. No support should be assumed from the face slab on the understanding that the perimetric joint may have opened, and no support from the rockfill should be allowed for, since significant displacement into the rockfill would be necessary to mobilize the resistance.

The toe slab must be stable against sliding and overturning. A proper assessment of the sliding friction angle should be made depending on the rock type, orientation of weak planes etc., not just a check against some arbitary 'sliding factor' or assumed friction coefficient.

It may be necessary to install additional grouted dowels, prestressed rock anchors, or buttress concrete downstream to maintain stability.

If the toe slab is high, a further problem which may occur is that excessive settlement of the face slab may occur and disrupt the perimetral joint. Problems were encountered in Golillas Dam due to such movements (in this case the valley sides were near vertical), (Amaya & Marulanda 1985). Particular attention should always be paid to compacting the rockfill near the toe slab, and finer rockfill compacted in thinner layers may be adopted (Fitzpatrick et al. 1985). However, one must also be careful not to obtain a modulus significantly higher than the rockfill or differential settlement may still occur.

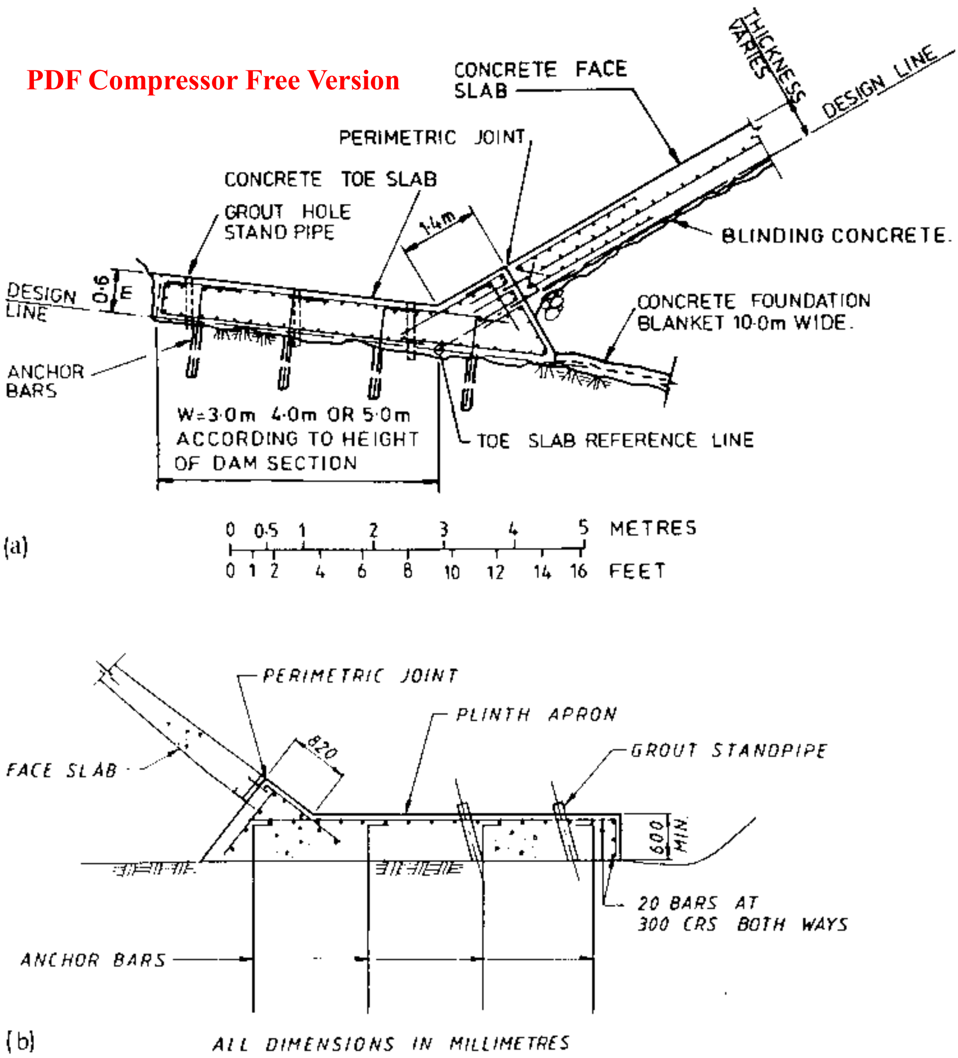
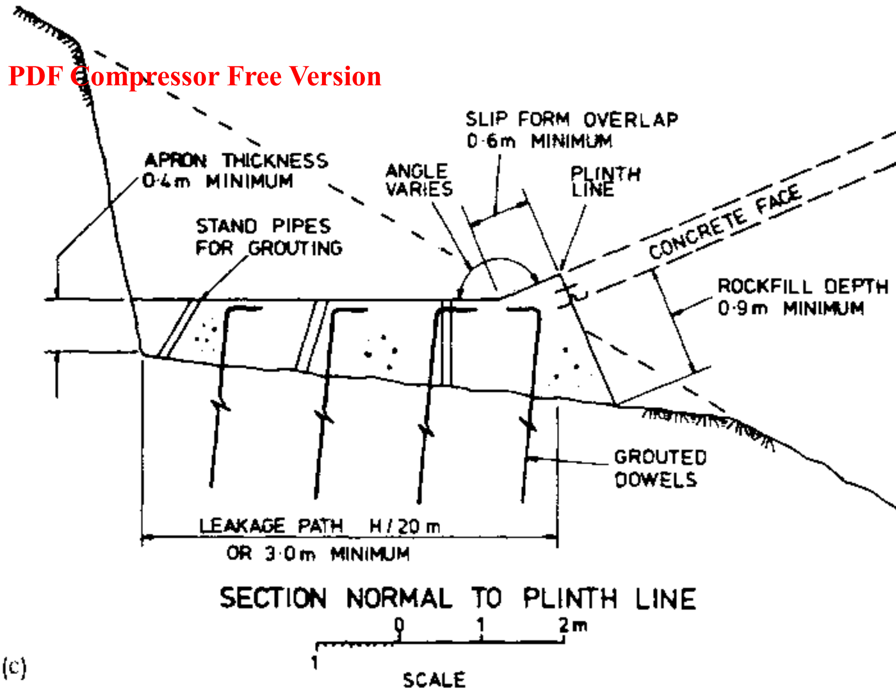


Figure 16.8. Toe slab designs for a) Mangrove Creek Dam, b) Boondooma Dam (MacKenzie & McDonald 1985, Rogers 1985).

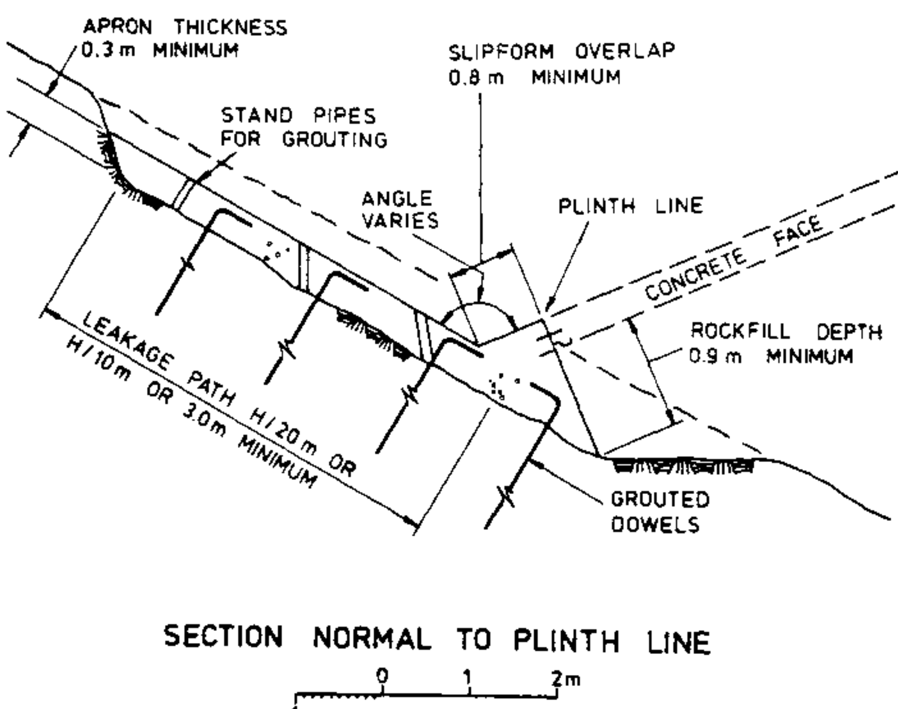
To ensure that the face slab displaces normal to its plane, and is not subject to bending, the Hydro-Electric Commission (Fitzpatrick et al. 1985) require a minimum of 0.9 m of rockfill under the face slab, see Figure 16.9c.

The toe slab is laid out as a series of straight lines selected to suit the foundation and topography. The angle points are not related to the vertical joints in the slab, and will be a compromise between added excavation and face slab; the slab thickness and simplicity of construction.

The toe slab may be laid out to be horizontal in a section normal to the centrelines of the plinth (as for Cethana Dam, Fig. 16.9c), or at right angles to the dam axis (as in Lower Pieman, Fig.



(c)



(d)

Figure 16.9. Toe slab designs for c) Cethana Dam, d) Lower Picman Dam (Fitzpatrick et al. 1985).

16.9d). The latter gives a smaller volume of excavation for the plinth, but concreting and grouting costs were increased due to more difficult forming and access for drill rigs. Varty et al. (1985) indicate that the HEC have reverted to the Cethana type layout for the 80 m high Crotty Dam. This layout was also adopted for the Foz do Areia Dam because, although excavation was increased this was compensated by better control of rockfill under the perimetric joint, a more straightforward concreting scheme and simpler drilling of grout holes (Pinto et al. 1985).

The toe slab is reinforced to control cracking due to temperature, and to spread out and minimise cracks which may tend to develop from any bending strains from grouting.

ICOLD (1989a) and Cooke & Sherard (1987) indicate that a single layer of steel, 100 to 150 mm clear of the upper surface, with 0.3% steel each way is adequate. Earlier designs had a lower layer of steel but the advantages of the added stiffness are outweighed by the difficulty of cleaning the rock prior to placing the concrete. Cooke & Sherard (1987) indicate that longitudinal reinforcing steel should be carried through construction joints, rather than using formed joints with water stops.

16.3.2 *Face slab*

16.3.2.1 *Face thickness*

The face slab thickness is determined from past experience. ICOLD (1989a) and Cooke & Sherard (1987) recommend that:

– For dams of low and moderate height (up to 100 m): Use constant thickness = 0.25 m or 0.30 m.

– For high and/or very important dams: Use thickness = 0.3 m + 0.002H where H = water head in metres.

These are minimum thicknesses; average thicknesses will be greater due to irregularities in the compacted Zone 2D. They assume a well constructed Zone 2D as detailed above. Earlier dams constructed on compacted rockfill were based on 0.3 m + 0.002H to 0.3 m + 0.004H, but improved construction methods have allowed a reduction to the above recommended values.

16.3.2.2 *Reinforcement*

Steel reinforcement is provided to control cracking due to temperature and shrinkage. In general the face slab is under compression.

ICOLD (1989a) and Cooke & Sherard (1987) recommend the use of 0.4% reinforcing steel in each direction, with possible reduction to 0.3 or 0.35% in areas of the slab which will definitely be in compression, while retaining 0.4% near the perimeter.

The reinforcing steel is placed as a single mat at or just above the centreline. The area of steel is calculated on the theoretical minimum thickness (Cooke & Sherard 1987, ICOLD 1989b). Fitzpatrick et al. (1985) indicate that at that time the HEC used 0.5% steel, based on the design thickness plus a 100 mm allowance for the added thickness, due to surface irregularity.

The reinforcing has generally been structural grade reinforcing steel (e.g. Australian Standard S230), but more recently high yield grade deformed bar (e.g. Australian Standard 410Y) has been used and is desirable if available. The percentage of steel is the same regardless of steel type.

Light reinforcement is provided in the face and toe slab at perimetric joints, and in some cases across vertical joints, to control spalling. Examples are shown in Figures 16.9a, 16.10 and 16.11.

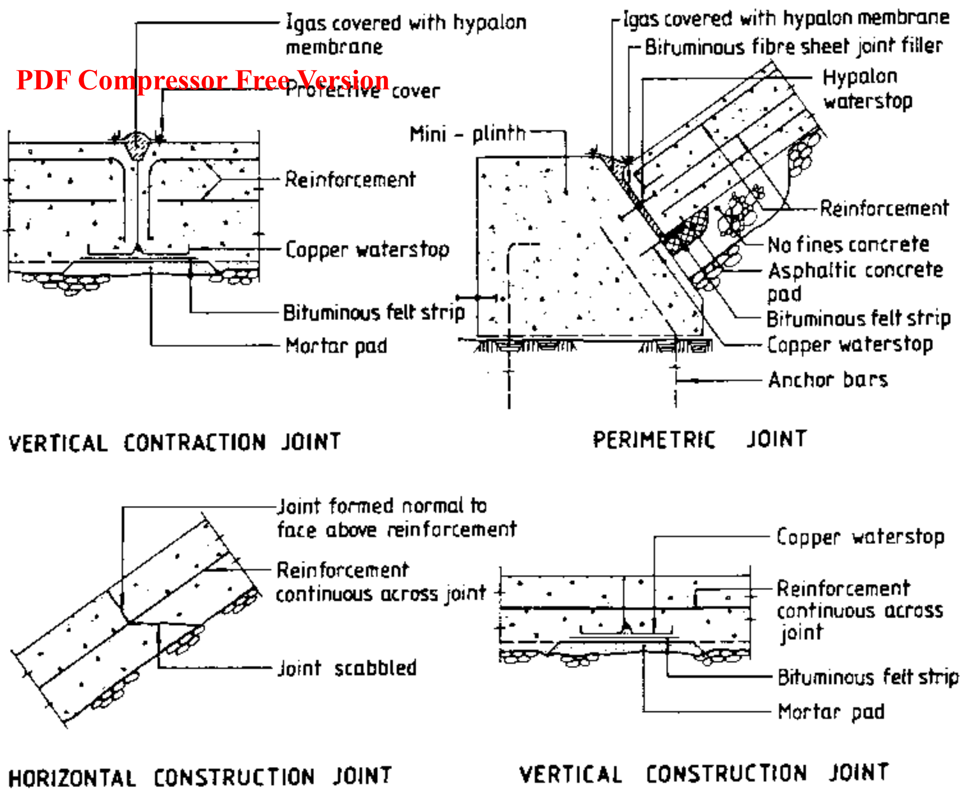


Figure 16.10. Joint details for Khao Laem Dam (Watakeekul et al. 1985).

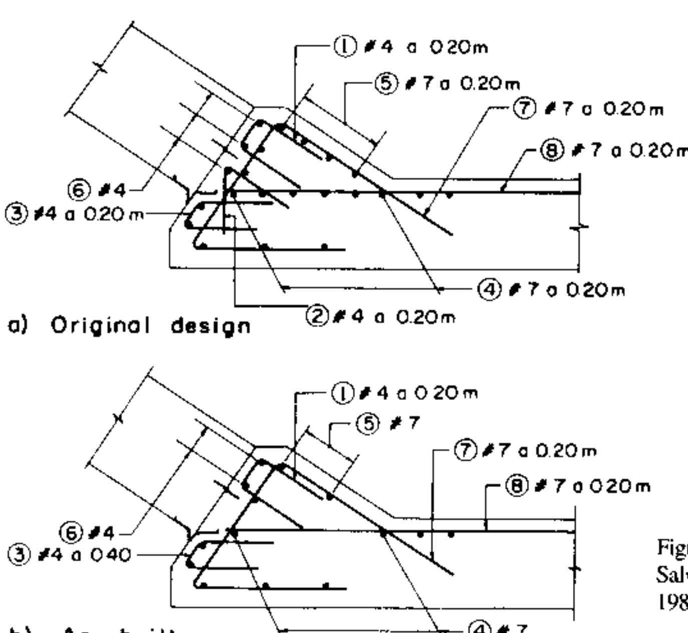


Figure 16.11. Antispalling steel in Salvajina Dam (Hacelas & Ramirez 1987).

16.3.2.3 *Vertical and horizontal joints*

ICOLD (1989a) indicates that present design practice does not include horizontal joints, except construction joints in which the reinforcing steel is carried through the joint. Details of such joints are shown in Figures 16.10 and 16.12. This was adopted because when horizontal joints with water stops were used it was difficult to obtain good quality concrete around the water stops, and some joints experienced spalling under compression and mild rotation.

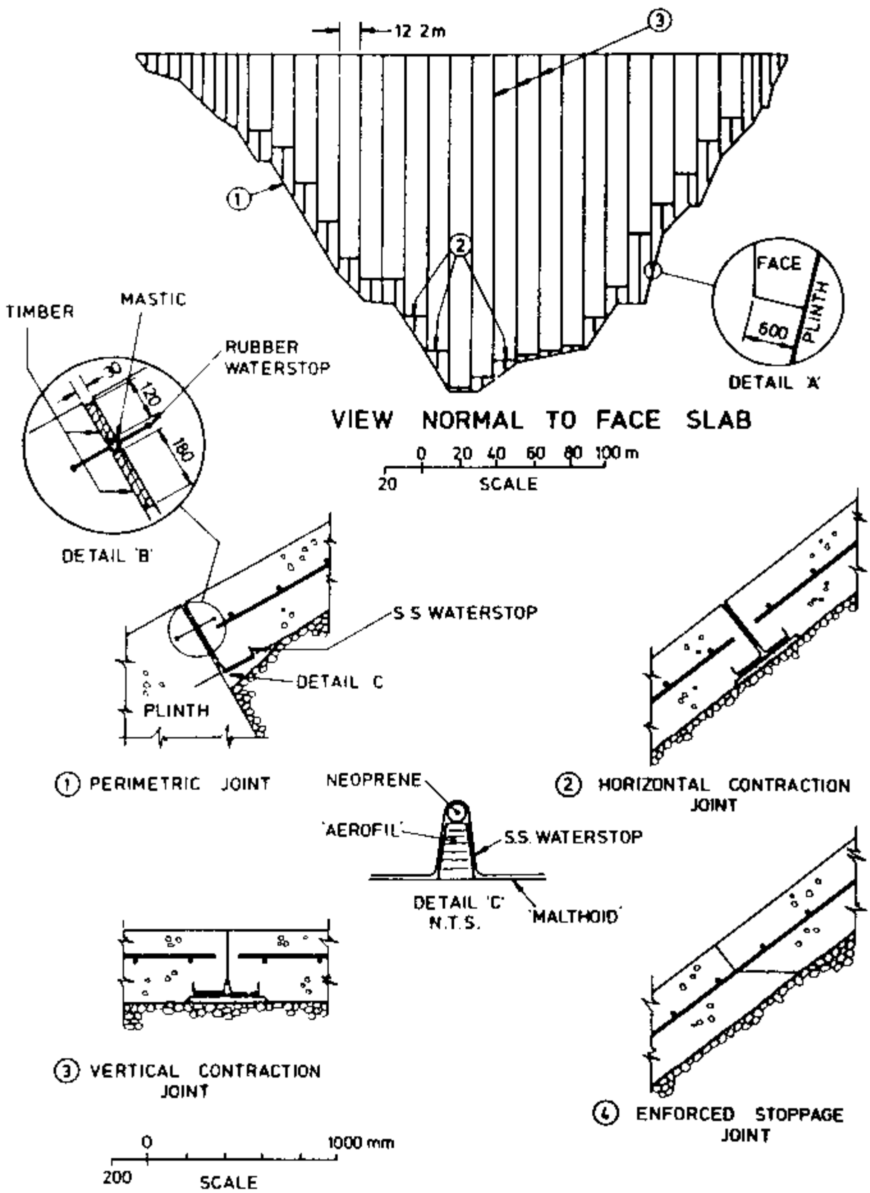


Figure 16.12. Joint layout and details for Lower Pieman Dam (Fitzpatrick et al. 1985).

The Hydro-Electric Commission retained a horizontal contraction joint for Lower Pieman Dam, to reduce thermal shrinkage and face cracking.

Vertical joints are generally provided at 12, 15 or 18 m spacing depending on construction factors. For smaller dams, narrower spacing is desirable, e.g. 6 m.

Most CFRD's have been constructed with each of the vertical joints being a construction joint as shown in Figures 16.10 and 16.12. The joints are painted with asphalt, not filled with a compressible filter as these have been shown to compress under load and cause opening of the perimetric joint. However, Cooke & Sherard (1987) advocate carrying the horizontal reinforcement through all but a few joints near the abutments, in the manner shown in Figure 16.10. This method gives a cost advantage and a reduction in potential leakage, by potentially eliminating water stops and anti- spalling steel.

Figure 16.13 shows the joint arrangement for the Foz do Areia Dam.

Beach (1987) indicates that the HEC have not had problems with vertical joints without reinforcing steel passing through them and suggests either method is satisfactory.

16.3.3 *Perimetric joint*

16.3.3.1 *General requirements*

Instrumentation of CFRD's has indicated that compressive strains develop in more than 90% of the face slab due to settlement of the rockfill.

When the reservoir is filled there is further displacement of the face slab, which leads to closing of vertical joints over most of the slab, and opening of the perimetric joint and those joints near the abutments. The face slab also pulls away from the toe slab, and offsets normal to the face slab, and parallel to the joint due to shear movement of the face. The joint is a common cause of leakage if not well designed, constructed and inspected.

Table 16.7 summarizes maximum perimetric joint movements measured on a number of CFRD's.

To accommodate these movements, joints with multiple water stops are provided. Figures 16.10, 16.12, 16.13, 16.14 and 16.15 show some examples.

The design shown in Figure 16.12 for Lower Pieman Dam is typical of practice up till around the mid 1980's. The joint includes two water stops

– Primary: Copper or stainless steel 'W' or 'F' shaped.

– Secondary: Central 'bulb' water stop made of rubber, hypalon or PVC.

Cooke & Sherard (1987) indicate that this arrangement has performed adequately on dams up to 75 m high, where perimetric joint movements are generally relatively small. Fitzpatrick et al. (1985) indicate that in two dams, 39 and 26 m high, only the primary water stop was used because very small movements were expected.

More recently, particularly for higher dams, a third water stop has been included in the form of a mastic filler covered with a PVC or hypalon sheet. These are shown in Figures 16.10, 16.13 and 16.15 for Khao Laem, Foz do Areia and Salvajina Dams. Cooke & Sherard (1987) argue that, because of the difficulty of having concrete around the central 'bulb' water stop, they believe that a joint with the copper (or stainless steel) water stop underlain by asphalt impregnated sand or concrete mortar and the mastic type stop, is the preferable detail.

It will be noted that in the designs shown, a wood plank approximately 12.5 to 20 mm thick, or some other compressible filler of similar thickness, is placed between the face slab and toe slab to prevent concentration of stresses in the joint during construction and before reservoir filling.

Table 16.7. Properties of dam rockfills. Bodtman & Wyatt (1985). Maximum perimeter joint movements (ICOLD 1989a).

Dam	Year completed	Maximum height (m)	Rockfill type	Joint movements in millimeters		
				Normal to joint (opening)	Parallel to joint (shear)	Normal to face (settlement)
Foz do Areia	1980	160	Basalt, CR	23	25	55
Salvajina	1984	148	Gravels, CG	9.1	15.4	19.5
Alto Anchicaya	1974	140	Hornfels, CR	125	15	> 100
Khao Laem	1984	130	Limestone, CR	5 ?	–	8 ?
Golillas	1978	127	Gravels, CG	100	–	36
Shiroro	1983	125	Granite, CR	30	21	> 50
Reece	1986	122	Dolerite, CR	7	–	70
Cethana	1971	110	Quartzite, CR	11	7	–
Murchison	1982	94	Rhyolite, CR	12	7	10
Kotmale	1984	90	Charnockite, CR, Gneiss	2	5	20
Sugarloaf	1982	85	Sandstone, CR	9	24	19
Mackintosh	1981	75	Greywacke, CR	5	3	20
Bastyan	1983	75	Rhyolite, CR	4.8	–	21.5
Serpentine	1972	39	Quartzite, CR	1.8	–	5.3
Paloona	1971	38	Chert, CR	0.5	–	5.5
Tullabardine	1982	26	Greywacke, CR	–	0.3	0.7

CR = Compacted rockfill; CG = Compacted gravel.

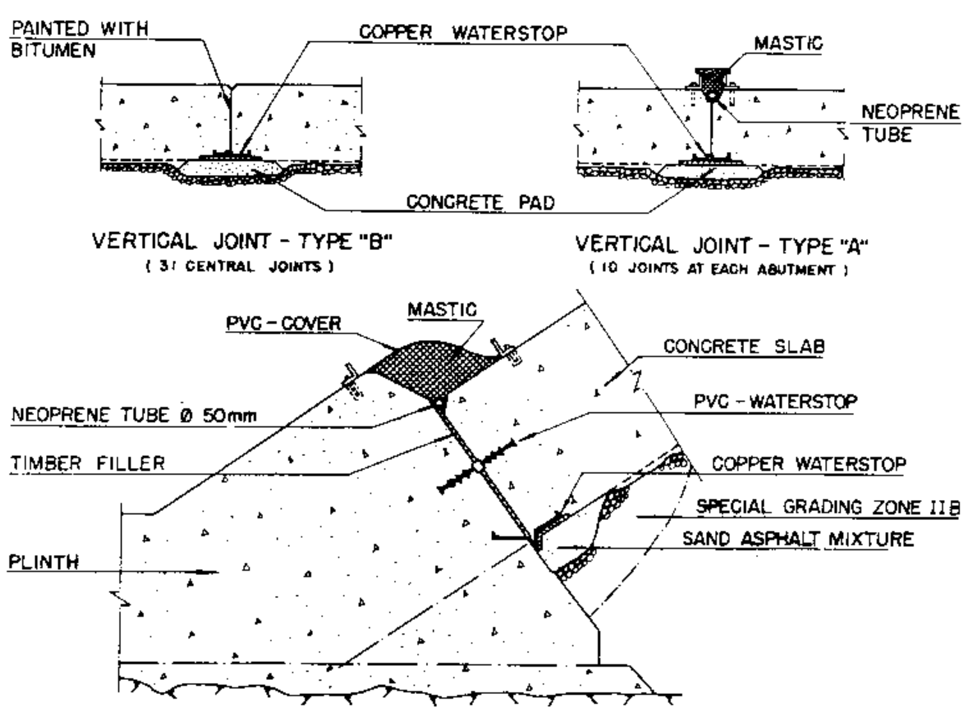


Figure 16.13. Joint details for Foz do Areia Dam (Pinto et al. 1985).

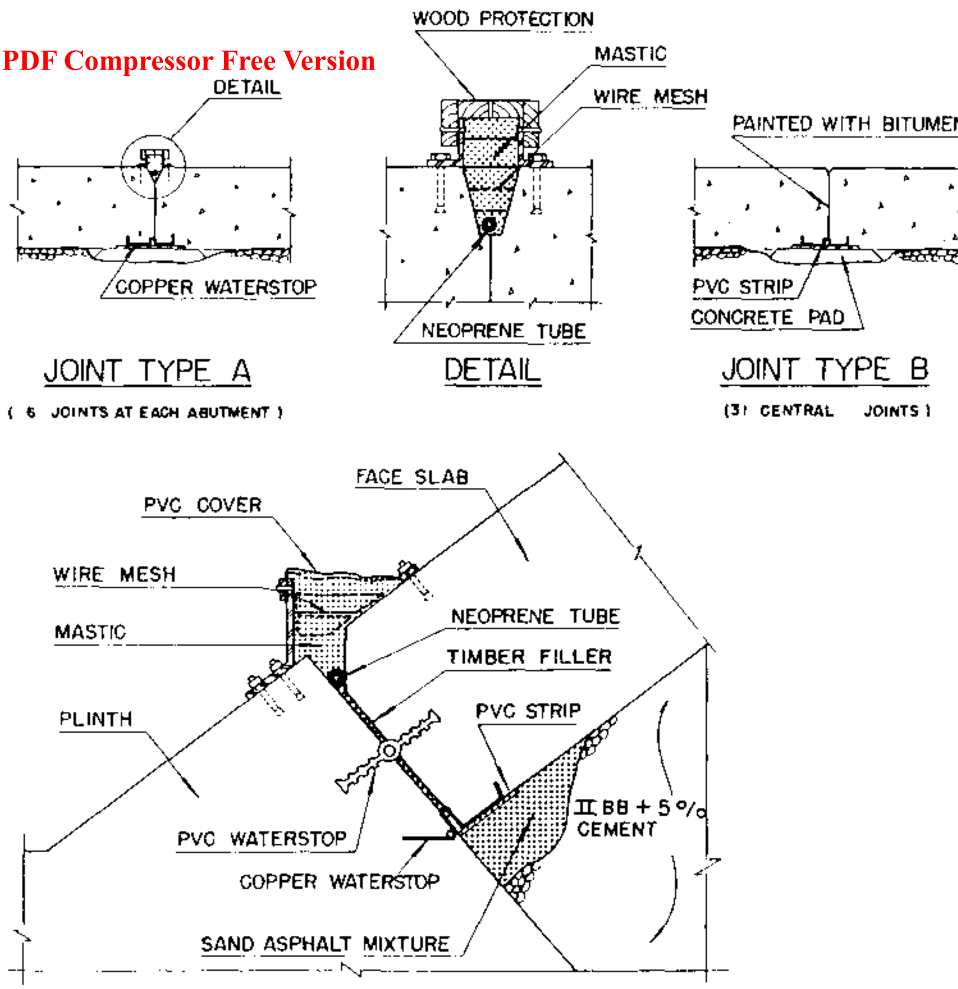


Figure 16.14. Joint details for Segredo Dam (Pinto et al. 1985).

16.3.3.2 *Water stop details*

Primary copper or stainless steel water stop. These are either 'W' or 'F' shaped, with a high central rib to permit shear movement between adjacent slabs. To prevent external water pressure from squeezing the rib flat, it is filled with a neoprene insert (12 mm dia. held in place with a strip of closed cell polythene foam 16 × 12 mm (Fitzpatrick et al. 1985). The water stop is supported on a cement mortar or asphalt impregnated sand pad. Fitzpatrick et al. (1985) indicate that the HEC seat the water stop on a 400 mm wide strip of tar impregnated felt ('malthoid').

Whether copper or stainless steel is used depends on the aggressive nature of the reservoir water, but also seems to be a matter of individual designer preference, with copper being more common. Fitzpatrick et al. (1985) indicate that for the Lower Pieman Dam the HEC departed from its earlier practice of using copper water stop (annealed after forming to give maximum ductility) to 0.9 mm thick grade 321 stainless steel. This was done because it was considered

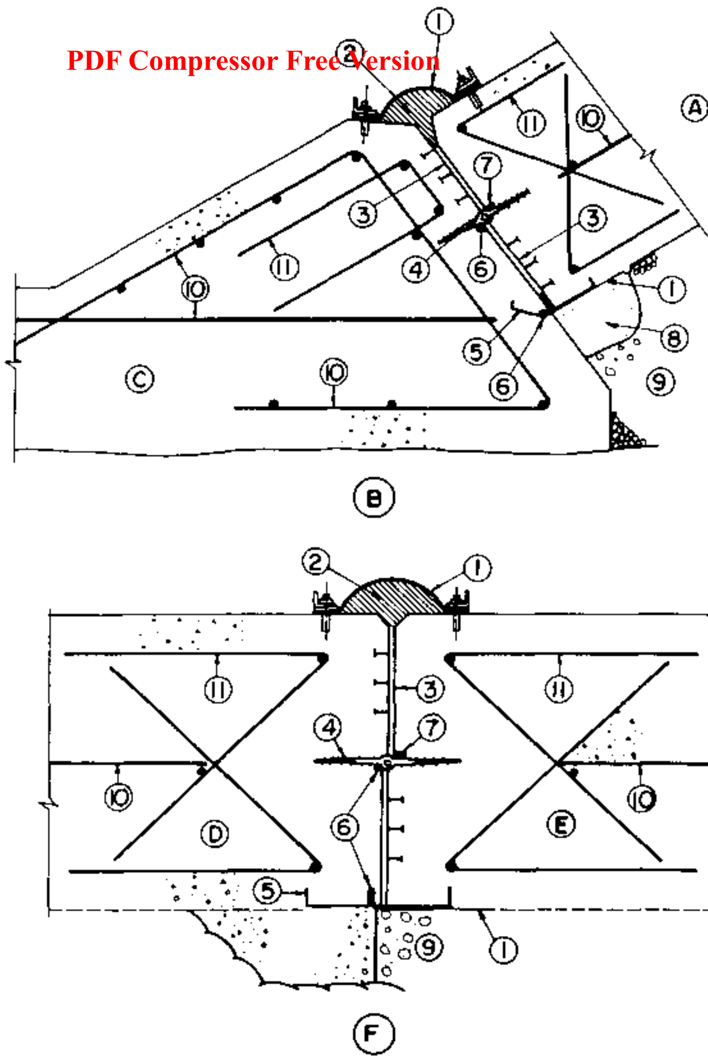


Figure 16.15. Salvajina Dam perimeter and near abutment vertical joints (ICOLD 1989a). (1) Hypalon band, (2) Mastic filler, (3) Compressible wood filler, (4) PVC waterstop, (5) Copper waterstop, (6) Neoprene cylinder, (7) Styrofoam filler, (8) Sand-asphalt mixture, (9) Zone 2, (10) Steel reinforcement, (11) Steel reinforcement to protect concrete against crushing and to protect waterstop. (A) Face slab, (B) Perimetric joint, (C) Toe slab, (D) Toe slab, (E) Face slab, (F) Abutments.

that the stainless steel would be 'more robust' during construction, and there was not a significant cost differential.

ICOLD (1989a) indicate that it is advisable to form the copper or steel water stops in continuous strips to minimize the need for field splices. They recommend use of an electrode of high fluidity (silver content greater than 50%) for welding copper waterstops to ensure full penetration into the two copper plates, then checking with a spark tester to ensure a good joint has been achieved. Fitzpatrick et al. (1985) indicate that for stainless steel, jointing consists of a lap joint fixed by spot welding, then sealing by tungsten-inert gas welding, so that only one metal is involved.

Pinkerton et al. (1985) indicate that in their experience the limit of shear displacement of 'W' type primary water stops is around 7.5 up to 50 mm for a 230 mm centre bulb rubber water stop and 10 mm for a PVC waterstop.

Centre bulb water stop. These are constructed of PVC, natural rubber or hypalon. Fitzpatrick et al. (1985) indicate that they prefer hypalon rubber instead of natural rubber or PVC because: Natural rubber in the atmosphere must be protected from oxidation and ozonation by the addition of antioxidants and antiozonants which could leach out. They will last indefinitely below minimum operating level where permanently submerged, but may be a problem between minimum operating level and flood level; PVC contains plasticizers, some of which are known to leach out.

Pinkerton et al. (1985) indicate a preference for hypalon rubber because it can accept much larger deformations as detailed above.

Mastic filler water stop. The concept of the mastic filler is that as the perimetric joint opens it will be forced into the opening by the water pressure. The mastic is covered with a PVC or hypalon membrane held in place by steel angles anchored to the concrete.

ICOLD (1989a) indicate that a chicken wire mesh is embedded in the mastic to prevent its flow downwards along the inclined joints. The covering membrane is convex upward to provide for enough mastic volume. It is important that the membrane be sealed effectively so that the water does not leak past the membrane, relieving the differential pressure needed to force the mastic into the crack. Adhesion is improved by painting the joints with mastic.

Cooke & Sherard (1987) indicate a preference for the membrane to be hypalon, not PVC (at least for higher dams), because of its proven 10 to 20 year life exposed to the weather. After this period the cover is not critical because joint opening should have ceased, and the mastic is wedged tightly into the joint, stopped against the other water stops, and if they have ruptured, against the mortar or asphalt impregnated sand pad.

The mastic which has been used is IGAS which is a bitumen compound. It retains its flow characteritics provided it is not exposed to sunlight for extended periods. This did occur in Golillas Dam (Amaya & Marulanda 1985) where the PVC and IGAS were exposed for about four years before the reservoir was filled.

16.3.4 *Crest detail*

It is common to provide a reinforced concrete retaining wall (wave-wall) at the crest of the dam to reduce the volume of rockfill.

Figures 16.16, 16.17 and 16.18 show details which have been adopted for Khao Laem, HEC dams, Golillas Dam and Macaqua Dam.

The base of the wall is usually above full supply level, and the wall is joined to the face slab with a flexible joint, such as that shown in Figure 16.19. The joint should be vertical, not normal to the plane of the slab, so that differential settlement can be accommodated.

The crest width will depend on operational requirements but may be as narrow as 4.9 m as shown in Figure 16.17. The crest wall is constructed after the face slab, giving a relatively wide platform on which to work while the face slab is under construction.

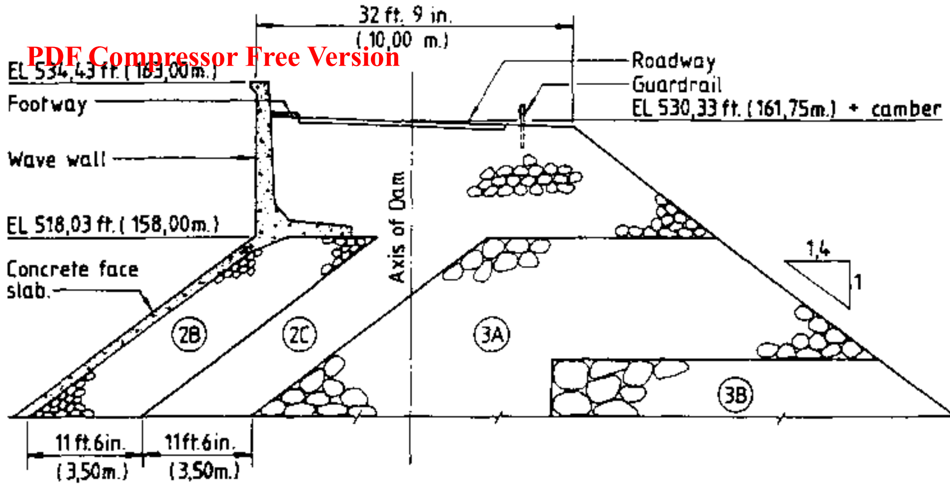


Figure 16.16. Crest detail, Khao Laem Dam (Watakeekul et al. 1985).

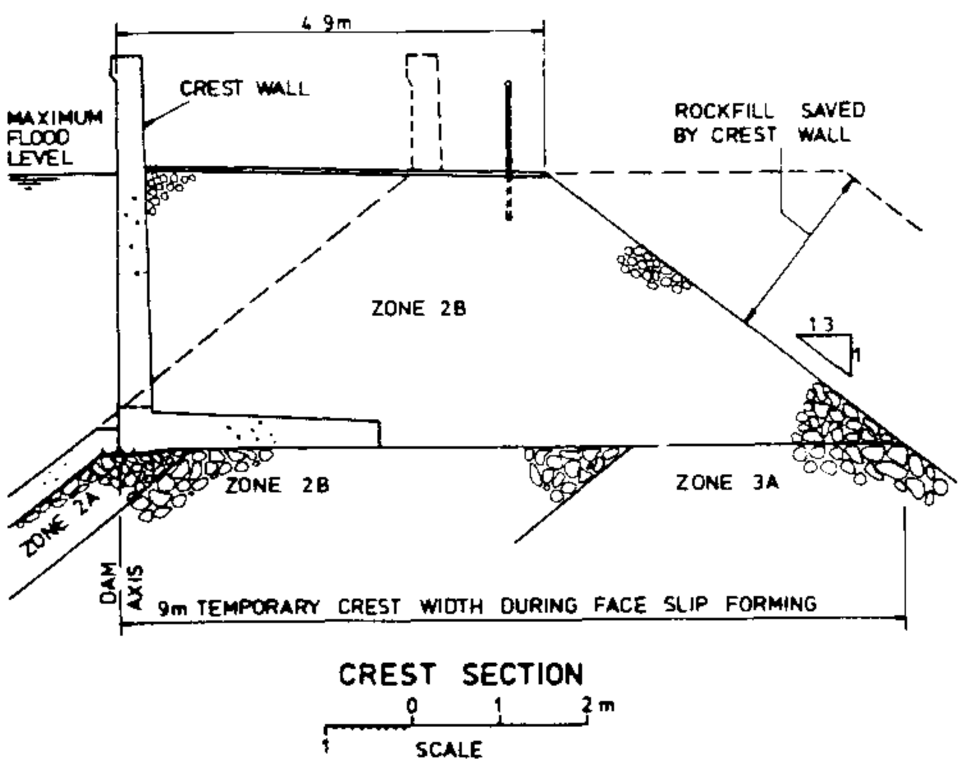


Figure 16.17. Crest detail adopted for Hydro-Electric Commission of Tasmania dams (Fitzpatrick et al. 1985).

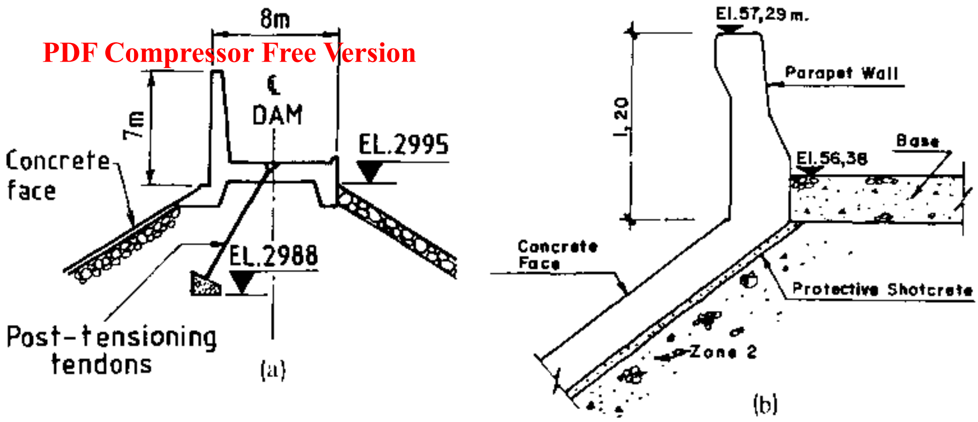
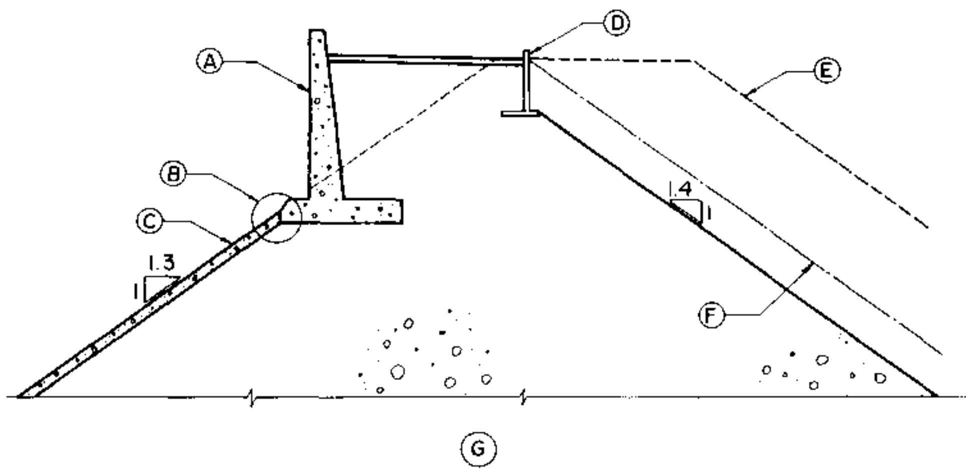


Figure 16.18. Crest details for a) Golillas Dam, b) Macaqua Dam (Amaya & Marulanda 1985, Prusza et al. 1985).



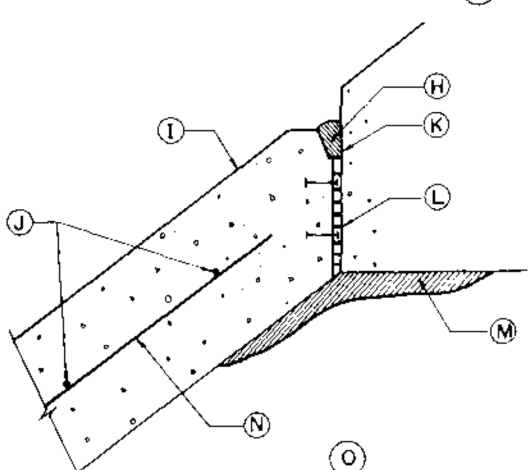


Figure 16.19. Crest details (ICOLD 1989a). (A) Upstream parapet wall, (B) See detail, (C) Concrete face, (D) Downstream parapet wall, (E) Additional rockfill if both parapet walls are omitted, (F) Additional rockfill if downstream parapet wall is omitted, (G) Crest details, (H) IGAS, (I) Concrete face, (J) Horizontal reinforcement, (K) IGAS primer, (L) Premoulded filler, (M) Mortar pad, (N) Inclined reinforcement, (O) Detail: Concrete face and upstream parapet wall joint.

16.4 CONSTRUCTION ASPECTS

16.4.1 *Rockfill trimming and rolling*

The upstream face of Zone 2D requires special compaction techniques, because the vibratory roller compacting the layers of fill cannot compact adequately on the edge of the fill. It is important that Zone 2D is well compacted to give a high modulus, uniform support for the face slab. It is also necessary to control loosening of rocks and erosion by rainfall runoff during construction (particularly adjacent to the toe slab where water is concentrated).

As described in ICOLD (1989a) and Fitzpatrick et al. (1985) the procedure generally adopted is:

– Trim the face to within 50 to 150 mm of the design plane. This may be done using an excavator working from the dam, guided by laser survey equipment.

– Compact with two to four passes of a 10 tonne steel drum roller without vibration. The roller is pulled by a winch cable up and down the slope.

– Compact with four to eight passes of the 10 tonne steel drum roller with the vibration applied when moving upslope only.

ICOLD (1989a) indicate that, before the final rolling, quick curing asphaltic emulsion at a rate of 2 to 4 litres/m^2, covered in fine sand is applied. Fitzpatrick et al. (1985) indicate that on earlier dams the HEC followed a similar practice (2 coat bitumen and chip seal), but that, for MacKintosh Dam, the stabilization was applied after rolling, and comprised of a single coat of bitumen emulsion with a cover of sand. Shotcrete has been used instead of bitumen emulsion, e.g. on Golillas and Salvajina dams 40 mm of shotcrete with size 9 to 19 mm in maximum size was adopted.

Phillips (1985) describes the use of a 1.4 by 0.9 m plate compactor in lieu of the vibratory roller on Batang Ai Dam. The technique is shown in Figure 16.20 and produced higher densities than could have been achieved with a 7 tonne roller.

The method has the advantage that small areas can be compacted and then protected (by shotcrete in this case), and this can be valuable in preventing erosion in high rainfall areas. ICOLD (1989a) indicate that the technique has also been used successfully on another dam.

16.4.2 *Face slab construction*

The concrete face slab is cast using a slip-form, except for the trapezoidal or triangular starter slabs adjacent to the toe slabs, which are screeded by hand methods ahead of the main face slab to provide a starting plane for the slip-form. These are usually half the width of the main slabs.

The slip-form is the full width of the panel. For a 1.3 m screed width, the form can move at 2 to 3 m per hour placing 60 mm slump concrete (Cooke 1984). Higher slump concrete requires a wider screed. Figure 16.21 shows the slip form used for Khao Laem Dam.

Varty et al. (1985) gives details of steel placement and other construction factors, as practiced on HEC dams.

Concrete is delivered to the form by bucket, pumping or in a chute.

The concrete used is typically specified as between 20 and 24 MPa at 28 days, (ICOLD 1989a), although higher strengths are used, e.g. Cethana and Lower Pieman had average 40 MPa concrete (Fitzpatrick et al. 1985). (ICOLD 1989a) indicate that higher strengths are not desirable as more shrinkage cracking is likely. They suggest the use of air entrainment to enhance water tightness and durability. The soundness and reactivity of aggregates used for the concrete is important.

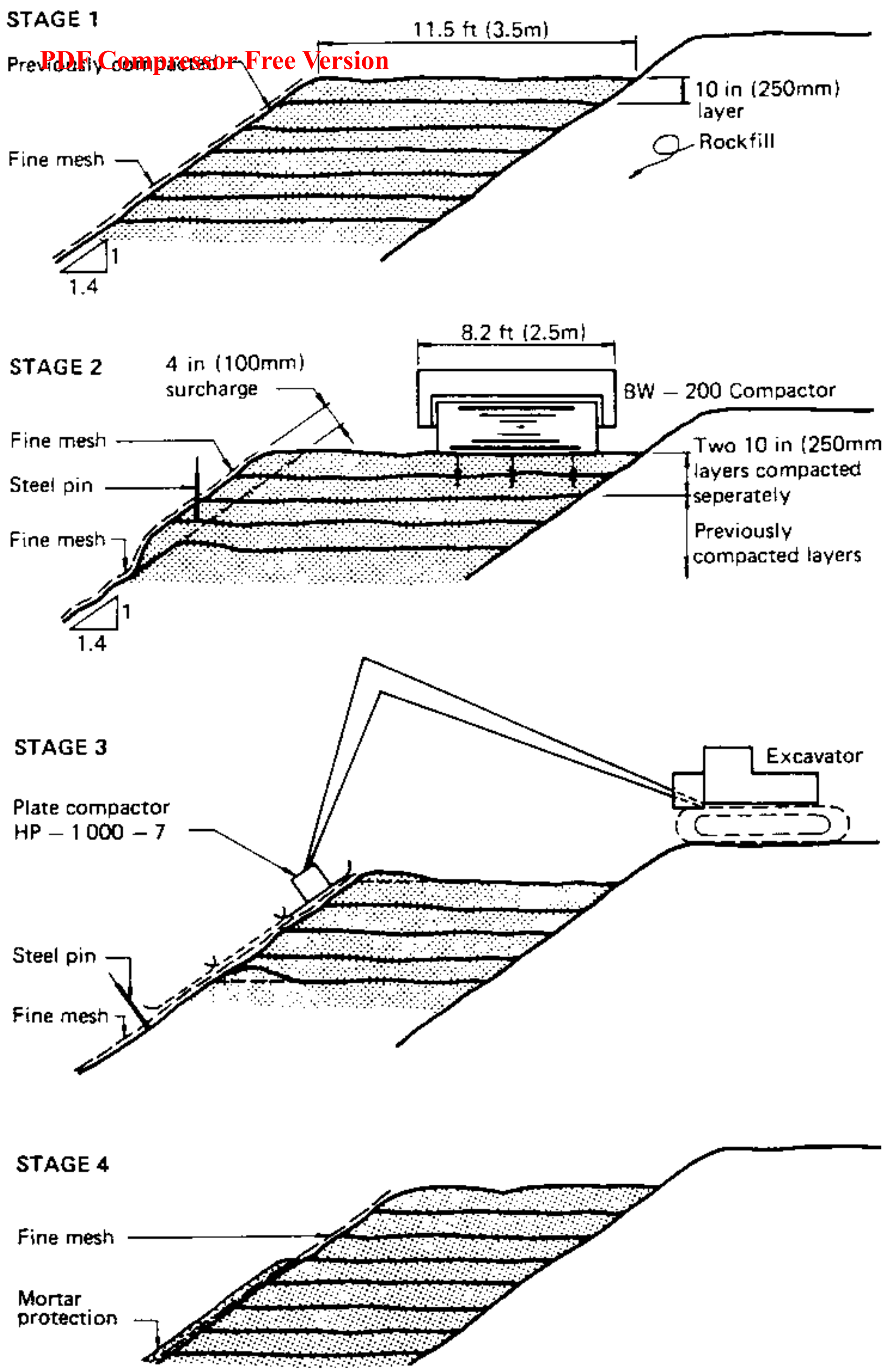


Figure 16.20. Face compaction at Batang Ai Dam (Phillips 1985).

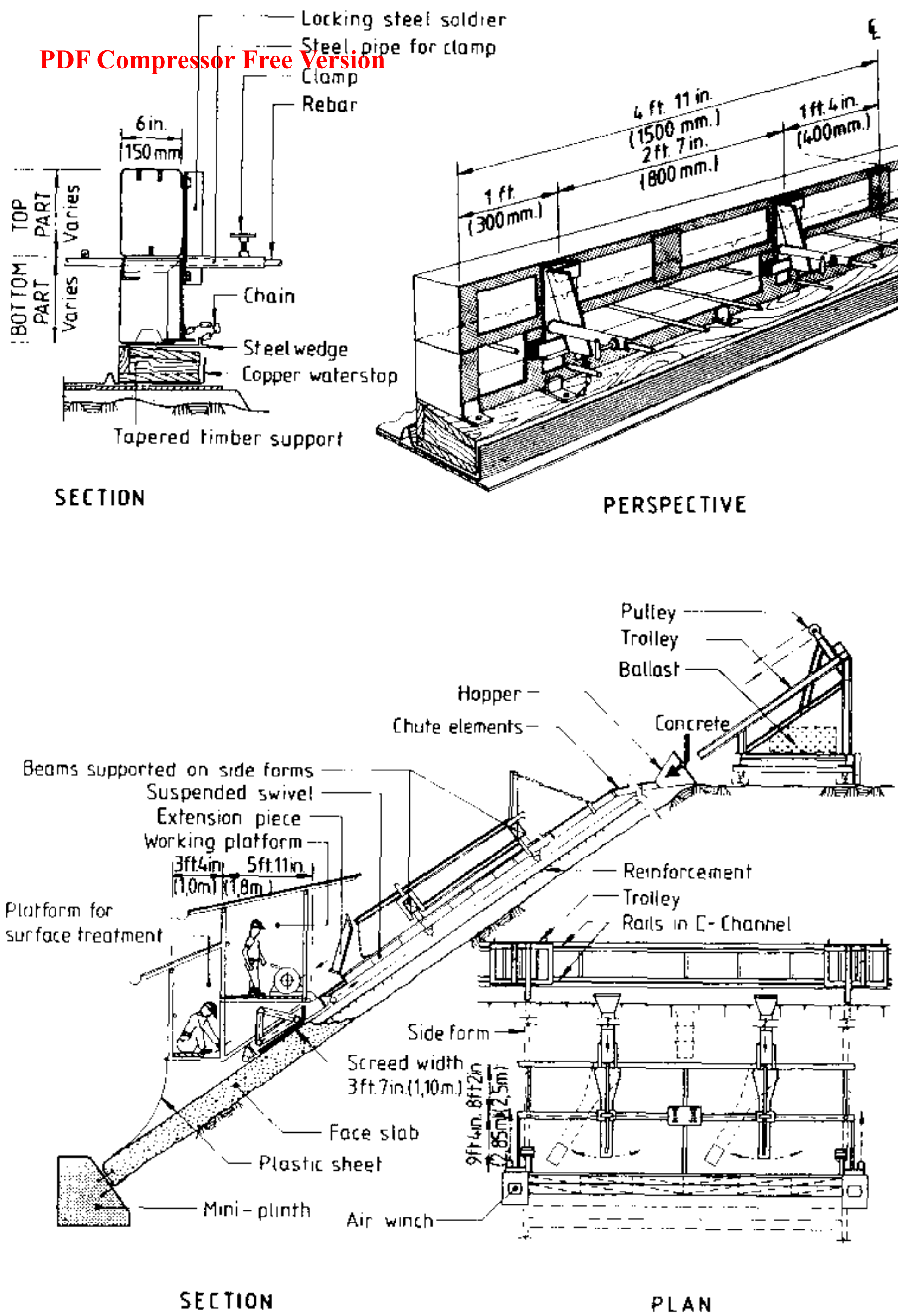


Figure 16.21. Slip-form for Khao Laem Dam face slab (Watakeekul et al. 1985).

The face slab is constructed on the face as it is presented. The face may have moved subsequent to trimming and compaction, due to the continual raising of the dam. The slab may therefore not end up being on a single plane, but the small variations do not affect performance or appearance.

The design thicknesses are minimum values, and average thicknesses are likely to be 50 to 75 mm greater. Early practice was to require that the face slab not be constructed until the rockfill placing was virtually completed so as to minimise post construction movements. More recently it has been shown that staging of the concrete face slab is acceptable, e.g. Salvajina and Foz do Aeria dams (Sierra et al. 1985, Pinto et al. 1985).

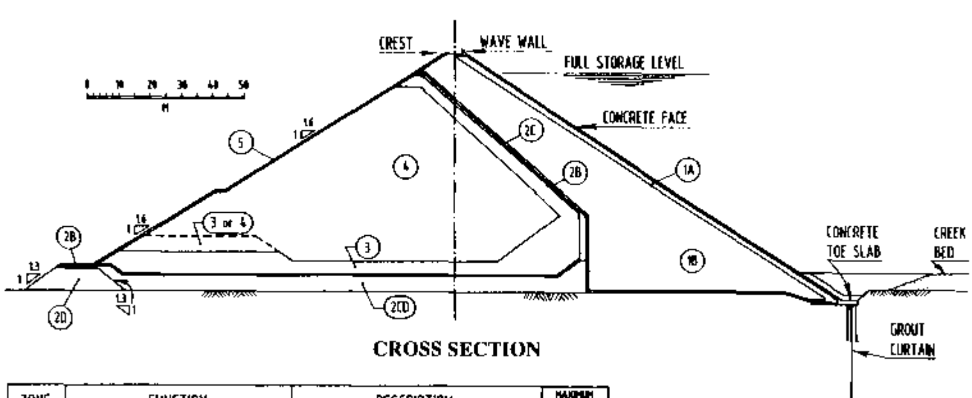
16.5 SOME NON-STANDARD DESIGN FEATURES

16.5.1 *Use of 'dirty' rockfill*

When the available rockfill breaks down under compaction to form a rockfill with a large proportion of sand and silt/clay size particles, the resulting fill may have adequate modulus, but may not be free draining. Rocks which are likely to do this are sandstones, siltstones, shale, schists and phyllites.

If this is the case, CFRD can still be used but must be zoned to provide drainage layers behind the face slab and on the foundation.

Mangrove Creek Dam is an example of this design, as discussed in detail in MacKenzie & McDonald (1985). Figure 16.22 shows the zoning.



CROSS SECTION

ZONE	FUNCTION	DESCRIPTION	MAXIMUM SIZE MM
1A	Semi-pervious zone	Fresh siltstone	150
1B		Fresh sandstone and siltstone	450
2A	Filters		20
2B		Fresh basalt	75
2C	Chimney drain		300
2CD	Bottom drain		600
2D	Toe drain	Fresh basalt from 100mm to 1200mm	1200
3	Free draining layer	Fresh sandstone and/or siltstone	600
4	Random zone	Fresh or weathered sandstone and siltstone	450
5	Downstream slope protection	Fresh sandstone from 600mm to 1200mm	1200

Figure 16.22. Mangrove Creek Dam (MacKenzie & McDonald 1985).

Other examples are Kangaroo Creek and Little Para Dams (Good et al. 1985) and Salvajina Dam (Sierra et al. 1985). Zoning for Salvajina Dam is shown in Figure 16.4.

In this case the rockfill zones are compacted gravel which have a significant silt content, and the drainage layer 2A is provided to ensure dam leakage will not result in build up of pore pressure in the dam.

It is not uncommon to zone the rockfill to ensure that the most free draining material is downstream of Zone 2E and on the base of the dam.

16.5.2 *Dams on erodible foundations*

Concrete face rockfill dams have been successfully constructed on foundations which have weathered seams or clay filled joints, which were potentially erodible under the high gradients which occur under the face slab.

In these circumstances, it has been necessary to prevent erosion by lengthening the seepage path and by providing filters into which the underseepage emerges in a controlled manner.

An example of such a dam is Winneke Dam. The deeply weathered siltstone rock precluded economic placement of the toe slabs on fresh rock and the slab was founded at a level where weathered rock contained infilled joints and crushed seams, some of which were dispersive clays (Figure 16.23).

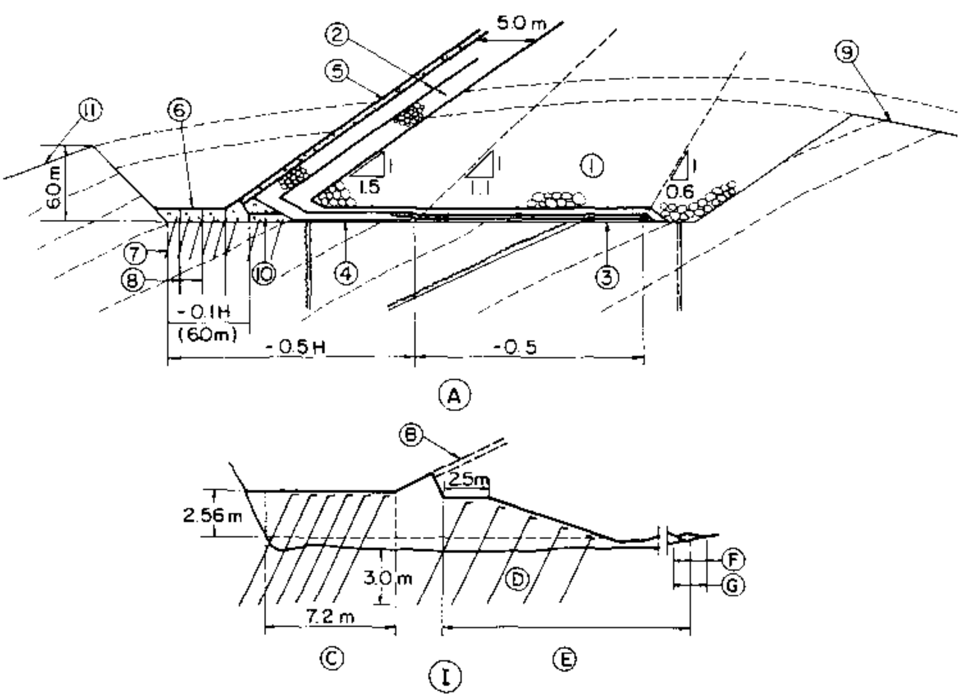


Figure 16.23. Winneke Dam foundation treatment (ICOLD 1989a, Casinader & Watt 1985). (A) Upstream toe detail, (B) Concrete face, (C) Anchor bars at 1 m centres longitudinally, (D) Foundation concrete 0.15 m thick, width 32.25 m, (E) Anchor bars at 2 m centres each way within thickened section, (F) Overlap 2 m minimum, (G) Fine filter 42 to 30 m, (H) Hydraulic head at foundation level, (I) High plinth with buttress. (1) Rockfill, (2) Transition, (3) Filters, (4) Foundation concrete, (5) Concrete face, (6) Plinth, (7) Anchor bars, (8) Grout curtain, (9) Foundation, (10), Buttress, (11) Original ground surface.

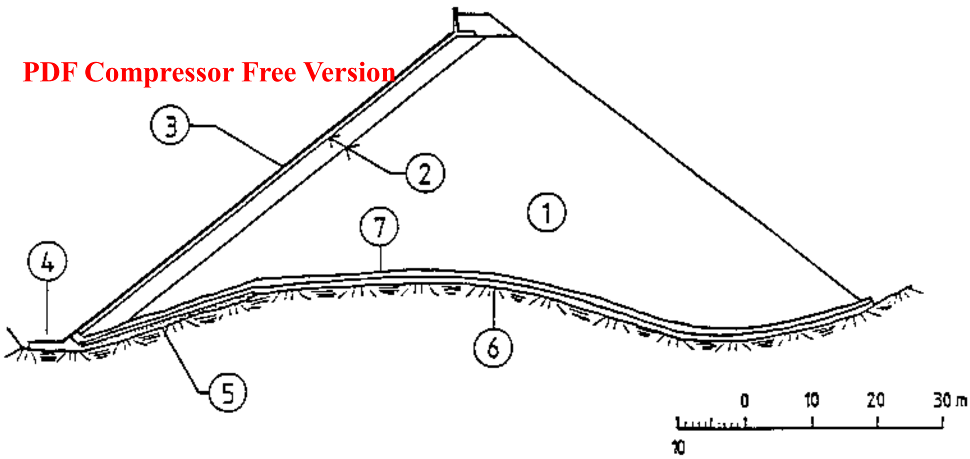


Figure 16.24. Foundation treatment and toe slab for left abutment of Lower Pieman Dam, (1) Rockfill zone, (2) Transition zones, (3) Concrete face slab, (4) Plinth, (5) Shotcrete blanket, (6) Fine filter, (7) Coarse filter (Li et al. 1991).

The measures adopted are described in Casinader & Watt (1985), Casinader & Stapledon (1979) and Stapledon & Casinader (1977) and included:

– The upstream toe excavation was taken down to the top of the highly weathered rock zone to reduce the number of infilled seams beneath the toe slab.

– Prior to grouting, seams in the grout holes were flushed out as far as practicable using air and water flushing

– The toe slab width adopted was 0.1H, or 6 m minimum, where H is the water head – wider than normally adopted to reduce gradients. The foundation downstream of the toe slab was blanketed with 150 mm of concrete, so that the total width of the toe slab and 'foundation concrete' was at least 0.5H.

– A filter was placed over the foundation for a distance 0.5H downstream of the foundation concrete, and the filter and foundation concrete covered by the transition layer to control erosion of fines from the foundation (even if the foundation concrete cracked).

The possibility of sliding movements on weak seams also necessitated a buttress on the toe slab, as shown in Figure 16.23.

Figure 16.24 shows similar design features for Lower Pieman Dam which was founded on deeply weathered schists with clay seams at cutoff level on the left abutment. In this case, abutment downstream of the plinth was covered with 150 mm layer of steel meshed shotcrete for a distance 0.5H downstream. The whole shotcrete layer, and the weathered rock between it and the downstream toe, was then covered by 2A and 2B type filter materials.

Another example of special foundation treatment is described by Sierra et al. (1985) for Salvajina Dam. In this case, the foundation was in part founded on residual soil, and wide concrete slabs were added upstream of the toe slab and filters downstream.

When gravelly alluvium or glacial soils are present in the river bed, these can commonly be left in place except for the first 0.3 to 0.5H downstream of the toe slab. This is dependent on confirmation that the gravels are of adequately high modulus and strength.

16.5.3 *Toe slab gallery*

The Khao Laem Dam, which was constructed on deeply weathered and in part karst foundations, incorporated a permanent gallery over the toe slab of the dam. Details are shown in Figure 16.25.

The value of the gallery, as a means of access for remedial work, was shown when leakage developed through the perimetric joint. The leak was reduced by grouting from the gallery, as described in Watakeekul et al. (1987).

Moreno (1987) describes the proposed incorporation of a grout and drainage gallery in a 190 m high CFRD, in a highly seismic area. Figure 16.26 shows the proposed gallery. Grouting is planned from the gallery to assist in meeting construction schedules.

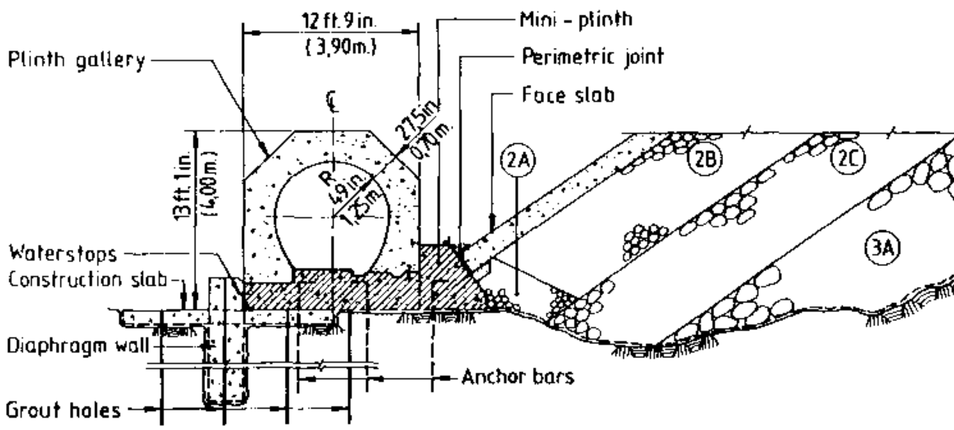


Figure 16.25. Toe slab and gallery, Khao Laem Dam (Watakeekul et al. 1985).

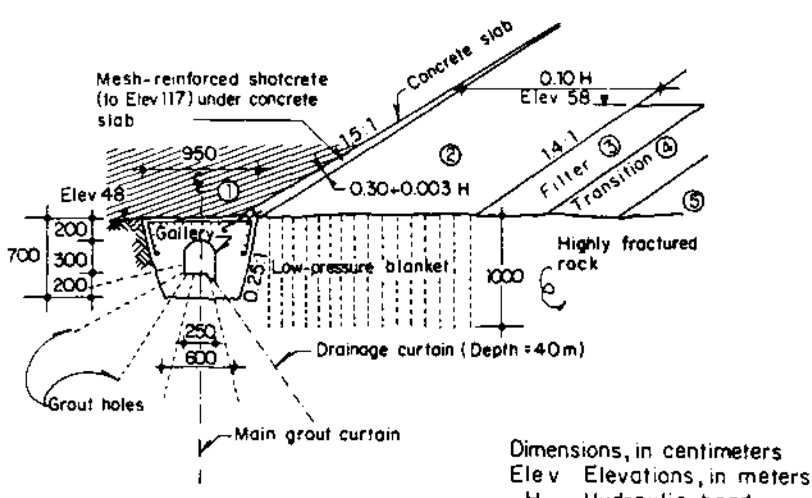


Figure 16.26. Proposed grouting and drainage gallery, Aquamilpa Dam (Moreno 1987).

16.5.4 *Earthfill cover over the face slab*

ICOLD (1989a), in their drawing showing typical design features of modern CFRD's (reproduced in Fig. 16.2), show earthfill and random fill placed over the lower part of the toe slab (Zones 1A and 1B).

Cooke & Sherard (1987) point out that these zones were first used on the lower part of Alto Anchicaya Dam, because at that time the dam height was breaking precedents. The detail has since been repeated on Foz do Areia Dam (Pinto et al. 1985); Khao Laem Dam (Watakeekul et al. 1985); Golillas Dam (Amaya & Marulanda 1985).

The concept is to cover the perimetric joint and toe slab in the lower elevations with impervious soil, which would seal any cracks or joint openings. Zone 1A is a minimum practical construction width, with Zone 1B provided for stability.

As pointed out by Cooke & Sherard (1987) many dams have been successfully constructed without this upstream zone, and if Zone 2D is graded fine to act as a filter to dirty fine sand in the event of leakage, there seems little justification for the fill in most dams. They do seem, however, to favour its application in the lower part of the toe slab in high dams.

16.5.5 *Spillway over dam crest*

Cooke & Sherard (1987) discuss the concept of building the spillway on the downstream face of a CFRD. They suggest that it is practical for ungated moderate size spillways with a peak discharge of around 250 to 30 m^3/sec per metre of chute width, and where flood flows are of short duration. They suggest the following principles:

– The whole of the rockfill should be compacted to Zone 3A standard to limit settlements.

– A layer of fine rock should be placed under the concrete, as for the face slab, and this should be rolled to make it a good even and stiff support for the concrete slab in the spillway.

– The spillway chute slab should be built from the bottom up, with continuous reinforcing in both directions, and extend into the side wall footings.

– For high dams, air grooves should be provided in the chute slab. These would also act as contraction joints. Contraction joints would be needed in the spillway walls.

The Hydro-Electric Commission have constructed such a spillway on the 80 m high Crotty Dam (HEC 1988).

The 12 m wide crest and chute have a discharge capacity of 210 m^3/sec. The chute has four 'hinges' in it to accommodate settlement, aeration slots to prevent cavitation and a flip bucket dissipator.

Namikas and Kulesza (1987) presented details of an emergency fuse plug spillway on a 32 m high CFRD. It is designed to operate for floods in excess of 1:1000 year return period.

Cooke & Sherard (1987) comment on the lack of precedents for spillways built on the dam embankment. Possibly the construction and operation of the Crotty Dam spillway will see further developments in this area.

CHAPTER 17

Mine and industrial tailings dams

17.1 GENERAL

This chapter is intended to give an overall appreciation of mine and industrial tailings disposal practice. It is written for those who have not had extensive experience in tailings disposal and serves as an introduction to the subject, rather than forming a complete basis for design of a tailings disposal system.

For those who are interested in more detailed information on mine tailings disposal, the following references are recommended:

– S.G. Vick (1983). *Planning Design and Analysis of Tailings Dams*, Wiley (an excellent book covering most aspects of tailings disposal).

– D.J.A. van Zyl and S.G. Vick (eds). *Hydraulic Fill Structures*. ASCE Geotechnical Special Publication No. 21. Colorado State University, August 1988 (includes many virtually state-of-the-art papers).

– International Commission on Large Dams (ICOLD). Technical Bulletin No. 45, *Manual on Tailings Dams and Dumps* (1982), and Technical Bulletin No. 44 *Bibliography, Mine and Industrial Tailings Dams and Dumps* (1982). (Respectively these give a reasonable basic overview and extensive bibliography).

– G.M. Ritcey (1989). *Tailings Management*, Elsevier (concentrates on chemical and metallurgical aspects of tailings disposal).

17.2 TAILINGS AND THEIR PROPERTIES

17.2.1 *What are mine tailings*

Mine tailings are the end product of mining and mineral processing, after the mineral has been extracted, or the unwanted material separated, e.g. shale and clays from coal; clayey fines from iron ore or bauxite. As shown in Figure 17.1, the process will usually involve crushing and grinding, leaching or separation, followed by dewatering or thickening before discharge to the tailings disposal area as a slurry.

Dewatering is usually carried out in a thickener, with the dewatered tailings being recovered from the discharge cone as shown in Figure 17.2.

The thickening process may be assisted by addition of flocculants and/or other chemicals, e.g. polyelectrolytes such as the Magnafloc range, and inorganic salts such as gypsum. The

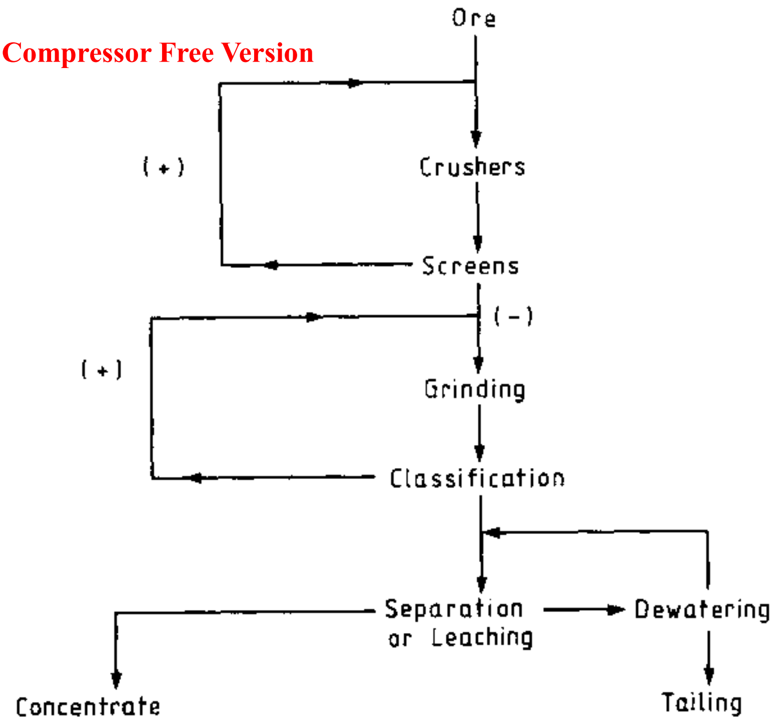


Figure 17.1. Procedures in mineral processing.

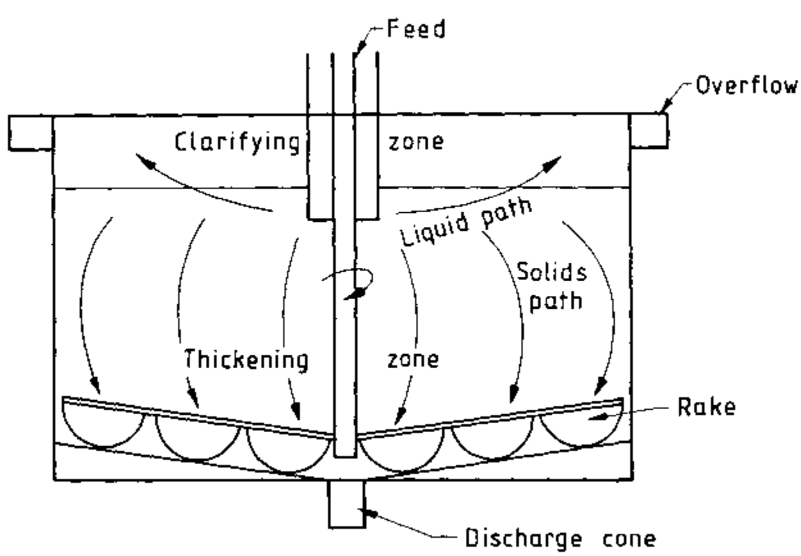


Figure 17.2. Minerals processing thickener.

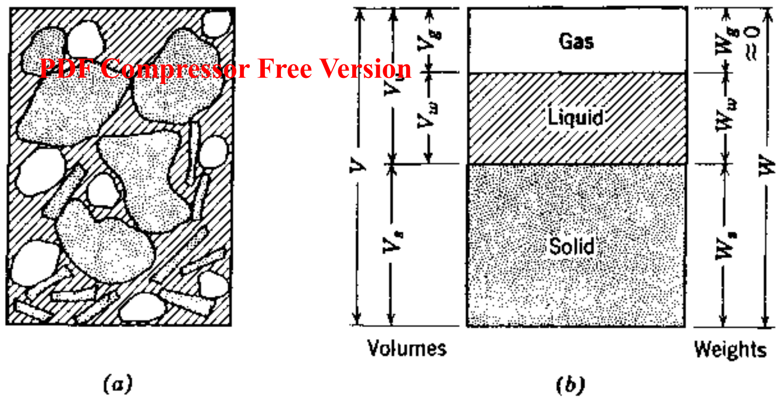


Figure 17.3. Tailings phase relationship, a) element of tailings with air filled and water filled voids; b) element separated into air, liquid and solid (Lambe & Whitman 1981).

polyelectrolytes are generally high molecular weight polyacrylamides, which have positive (cationic) or negative (anionic) charges. These adsorb to the electrically charged particles in the tailings, and form large flocs which settle more quickly in the thickener. Inorganic salts such as gypsum ($CaSO_4$) operate by cation exchange, with high charge density Ca^{++} cations replacing low charge density Na^+ cations, reducing dispersion effects and promoting flocculation.

This is discussed in more detail in Mitchell (1976) and Fell (1988). These chemicals affect the properties of the tailings discharged to the tailings disposal area and must be included in trial processes when testing tailings properties.

17.2.2 *Tailings terminology and definitions*

Most properties of tailings are described in soil mechanics terminology, but there are some terms which are in common usage which originate from mineral processing. Tailings are a mixture of solids, water (and air when not saturated), as shown in Figure 17.3, and the various terms used relate largely to the relative proportions of those present.

17.2.3 *Tailings properties*

17.2.3.1 *General*

Tailings properties vary considerably depending on the ore from which they are derived, the mineral process, whether the ore is oxidised (i.e. from weathered rock), etc. Table 17.1, taken from Vick (1983), gives general characteristics which are a reasonable indication of engineering behaviour. However, exceptions will occur, e.g. if a copper ore is oxidised it will yield 'slimes,' or clay sized tailings which may be quite plastic; oxidised uranium ore may also give plastic 'slimes.'

When planning any tailings disposal project, tailings representative of the operational mine should be tested to determine the properties required to predict behaviour in the storage. It should be noted that, in many cases, tailings produced from trial crushing and grinding in laboratory processing studies often are not truly representative of the operational process plant, giving different results to those which would be obtained on truly representative tailings. This

Table 17.1. Summary of physical tailings characteristics (Vick 1983).

Category	General character
Soft-rock tailings	
Fine coal refuse; trona insols; potash	Contain both sand and slime fractions, but slimes may dominate overall properties because of presence of clay
Hard-rock tailings	
Fine coal refuse; copper; gold-silver; molybdenum; nickel (sulphide)	May contain both sand and slime fractions, but slimes are usually of low plasticity to nonplastic. Sands usually control overall properties for engineering purposes
Fine tailings	
Phosphatic clays; bauxite red muds; fine taconite tailings; slimes from tar sands tailings	Sand fraction generally small or absent. Behavior of material, particularly sedimentation-consolidation characteristics, dominated by silt- or clay-sized particles and may pose disposal volume problems
Coarse tailings	
Tar sands tailings; uranium tailings; coarse taconite tailings; phosphate sand	Contain either principally sands or non-plastic silt-sized particles exhibiting sandlike behavior and generally favorable engineering characteristics

leads to the perception that laboratory tests do not accurately predict field behaviour. The authors' experience is that laboratory tests on representative samples can reasonably predict field behaviour.

17.2.3.2 *Particle size*

Figure 17.4 gives some examples of particle size distributions from coal, lead-zinc, and gold-silver tailings (from Vick 1983).

The present trend is towards grinding metalliferous ores finer than in the early 1970's, so the finer tailings shown in Figure 17.4 are more typical of what can be expected.

The particle size distribution for oxidised tailings and tailings from washeries (e.g. coal, iron ore, bauxite) also depends on the test method used and particularly on whether dispersants are added.

The standard soil mechanics test for determination of particle size of fine materials is to use a hydrometer analysis and for the standard test a dispersant (calgon, i.e. sodium hexametaphosphate) is added to break the particles to their constituent size. In the thickener, and as disposed to the tailings disposal area, the particles may remain flocculated. Figure 17.5 shows particle size distributions for tailings from two coal mines (Wambo, Hunter Valley; Riverside, Central Queensland); a bauxite mine (Weipa, North Queensland) and iron ore mines (Newman and Hamersley, Western Australia). Tests with and without dispersant show that the 'true' behaviour is to act as a silt-sand mix, whereas without dispersant added, the tailings have a high clay size fraction.

17.2.3.3 *Mineralogy*

The tailings' behaviour can often be related to the mineralogy of the constituent particles. To illustrate this, Tables 17.2 and 17.3 list mineralogy, chemistry and soil mechanics properties of several tailings. The following should be noted:

– The bauxite tailings from Weipa have a high proportion of amorphous clay minerals

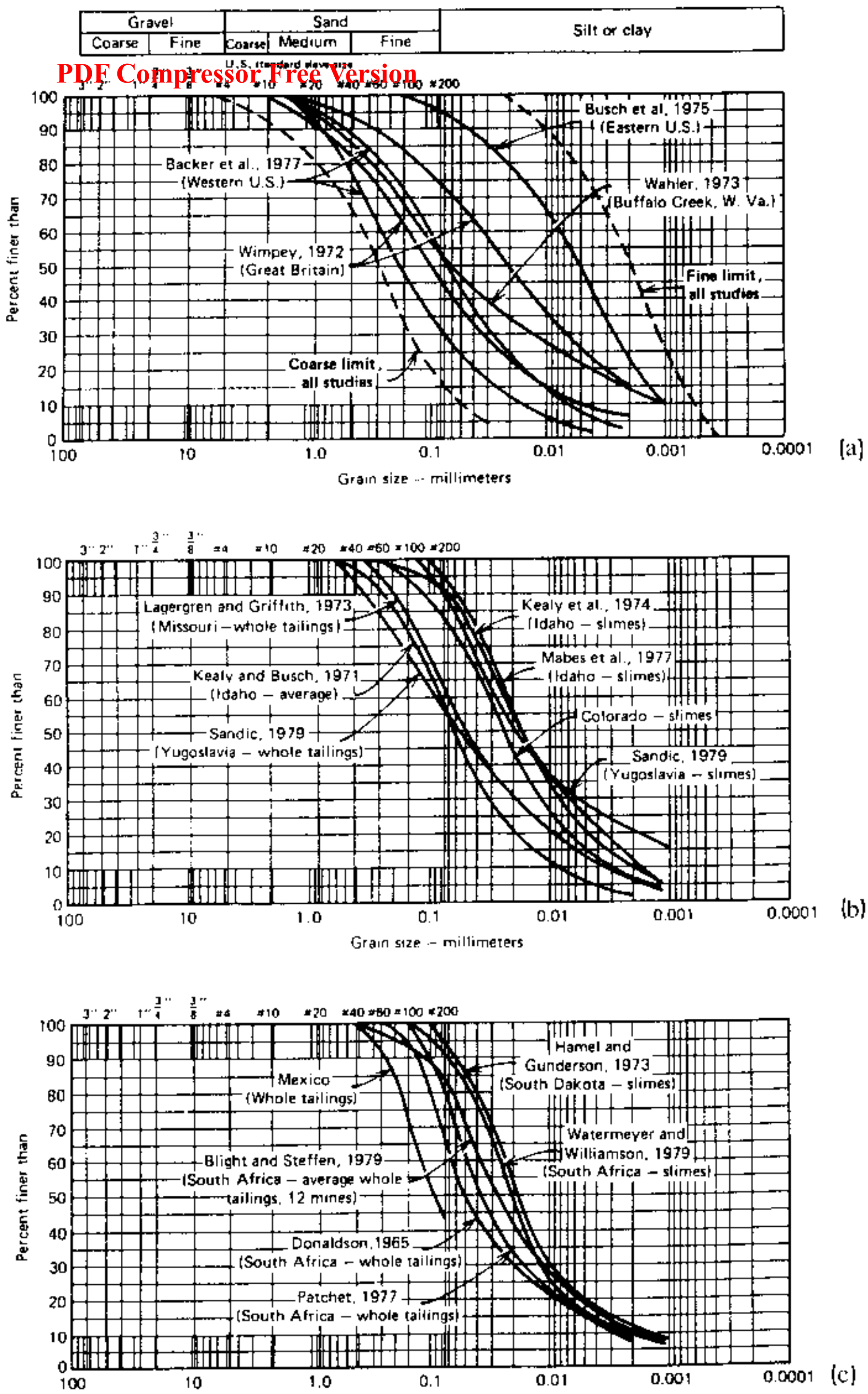
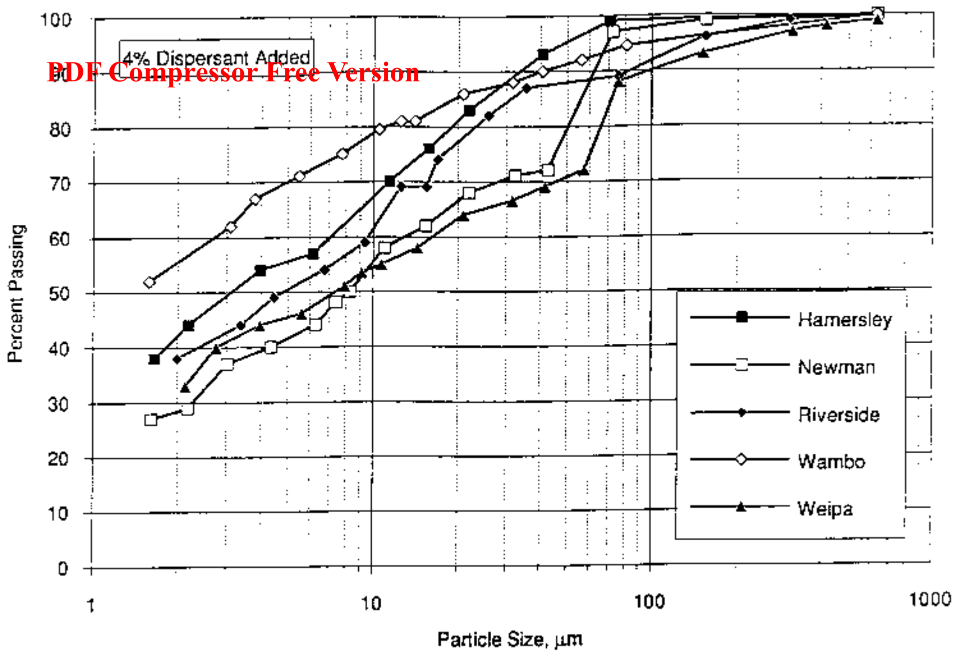


Figure 17.4. Typical tailings particle size distribution: a) coal washing; b) lead-zinc; c) gold-silver (Vick 1983).



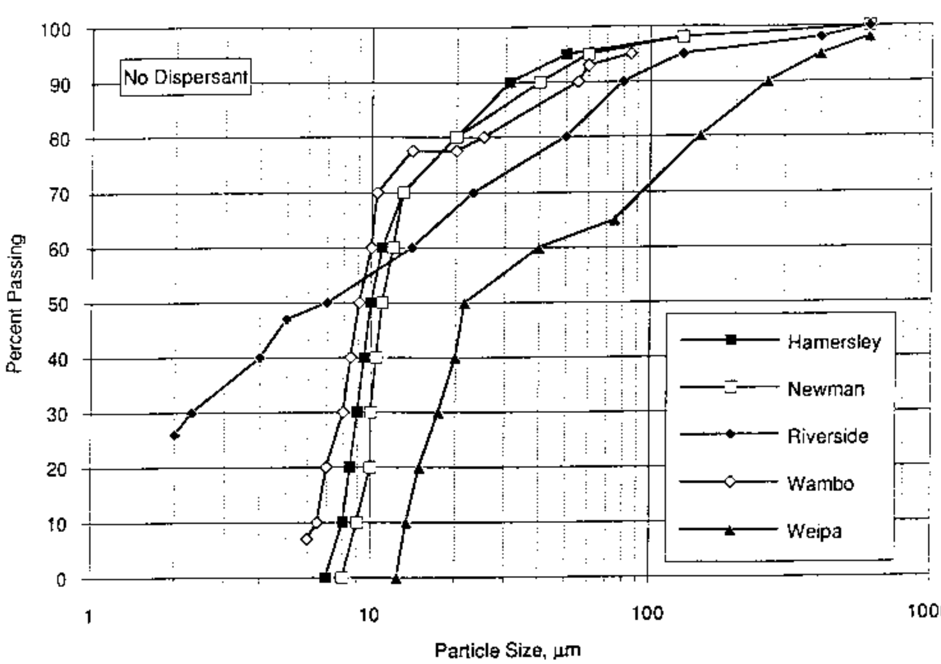


Figure 17.5. Particle size distributions of tailings with and without dispersants.

Table 17.2. Mineralogy and chemistry of some fine grained tailings.

Tailings	Mineralogy	Chemistry (% by weight of dried tailings) S_iO_2	Al_2O_2	Fe_2O_3	CaO	MgO	Na_20	K_2O	LOI (1)	pH tailings water
Weipa 1	Gibbsite (45%), boehmite (18%), kaolin (12%), quartz (17%), hematite (5%)	19	52	6	<0.1	<0.1	<0.1	<0.1	18	6.9
Weipa 2	Gibbsite (34%), boehmite (35%), kaolin (10%), quartz (8%)	10	53	12	<0.1	<0.1	<0.1	0.1	19	6.4
Wambo	Na and Ca montmorillonite, kaolin, coal, quartz (2)	28	12	1	0.9	1.0	0.8	0.4	51	6.8
Riverside	Illite, Ca montmorillonite, kaolin, quartz (2)	29	13	0.5	<0.1	0.3	0.3	0.7	54	7.4
Mt Newman	Hematite (40%), kaolinitic shale (50%), quartz (6%) (2)	17	12	63	0.4	0.7	<0.1	0.4	5	7.4
Hamersley	Hematite (34%), kaolinitic shales (52%), limonite (8%), goethite (19%) (2)	20	14	60	0.1	0.3	<0.1	0.1	7	7.1
North Kalgurli	Quartz, kaolin, gibbsite (2), host rock, dolerite, basalt, sulphide and oxide ores	55	14	13	2.3	1.1	2.8	2.2	7	7.9
Broken hill	Quartz (50%), rhodenite (20%), calcite (12%), garnet (6%), manganhedbergite (4%), pyrrhotite (4%)	56	7	12	6.7	1.1	1.1	1.3	1	7.7

Notes: (1) Loss on ignition (LOI) at 1000°; (2) Mineralogy percentages are approximate and vary with production sources.

Table 17.3. Soil mechanics properties of some fine grained tailings

Tailings	Water content (%)	Dry density (t/m^3)	% solids	Soil particle density (t/m^3)	Atterberg limits (%) Liquid limit	Plastic limit	Plasticity index	Particle size (%) Sand	Silt	Clay
Weipa 1	675	0.14	13	2.75	44	27	17	20	47	33
Weipa 2	362	0.25	22	2.85	43	26	17	15	45	40
Wambo	411	0.22	20	1.86	74	28	46	14	36	50
Riverside	250	0.32	29	1.74	44	28	16	16	50	34
Newman	192	0.46	34	3.70	33	22	11	20	60	20
Hamersley	169	0.50	37	3.50	30	21	9	5	55	40
North Kalgurli	71	0.94	59	2.81	28	21	7		Not available	
Broken Hill	462	0.20	18	3.05		Non plastic				

Note: (1) Particle size with dispersant added as per AS1289.

which, while fine grained, lead to a relatively low liquid limit and plasticity index and can be reasonably (for fine tailings) easily disposed.

– The coal washery tailings from Wambo have a high proportion of sodium montmorillonite, which leads to a very low settled density, high plasticity and tailings which are very difficult to handle.

– The coal washery tailings from Riverside, and gold tailings from North Kalgurli, contain clays which have intermediate activity between Weipa and Wambo and behave accordingly. The coal content of Wambo and Riverside tailings give a low specific gravity.

– The iron ore tailings from Newman and Hamersley have a significant haematite content, giving a high specific gravity. The clay minerals are not very active and, although fine grained, the tailings tend to dry readily.

– The Broken Hill tailings have been produced from high strength unweathered rock, and yield a nonplastic silt-sand mix which settles rapidly to a relatively high density.

Commonly used terms and their definitions include:

Void ratio

$$e = \frac{V_v}{V_s}$$

Degree of saturation

$$S = \frac{V_w}{V_v}$$

Porosity

$$n = \frac{V_v}{V} = \frac{V_v}{V_v + V_s}$$

Specific gravity (solids)

$$G = \frac{\gamma_s}{\gamma_w}$$

where V_v = volume of voids
V_w = volume of water
V_s = volume of solids
W_w = weight of water
W_s = weight of solids
M_w = mass of water
M_s = mass of solids

γ_s = unit weight of solids $= \dfrac{W_s}{V_s}$

γ_w = unit weight of water $= \dfrac{W_w}{V_w}$

Water content

$$w = \frac{W_w}{W_s} \qquad \text{(may also be defined as } W_w \;/\, (W_w + W_s) \text{ by some metallurgists)}$$

Pulp density (% solids)

$$P = \frac{W_s}{W} = \frac{1}{1 + w}$$

where $W = W_s + W_w$

Degree of saturation

$$S = \frac{V_w}{V_v} = \frac{\gamma_d\, w\, G}{G\gamma_w - \gamma_d}$$

Total unit weight (total density)

$$\gamma_t = \frac{W}{V} = \frac{G + Se}{1 + e}\ \gamma_w$$

Dry unit weight (dry density)

$$\gamma_d = \frac{W_s}{V} = \frac{G}{1 + e}\ \gamma_w = \frac{\gamma_t}{1 + w}$$

17.2.3.4 *Dry density and void ratio*

Table 17.4 gives some typical in-place dry densities and void ratios. The lower void ratios/

Table 17.4. Typical in-place densities and void ratio (adapted from Vick 1983).

Tailings type	Specific gravity	Void ratio	Dry density (tonnes/m^3)
Fine coal refuse			
Eastern US	1.5-1.8	0.8-1.1	0.7-0.9
Western US	1.4-1.6	0.6-1.0	0.7-1.1
Great Britain	1.6-2.1	0.5-1.0	0.9-1.35
Oil sands			
Sands	–	0.9	1.4
Slimes	–	6-10	
Lead-zinc			
Slimes (1)	2.9-3.0	0.6-1.0	1.5-1.8
	2.6-2.9	0.8-1.1	1.3-1.65
Gold-silver			
Slimes	–	1.1-1.2	
Molybdenum			
Sands	2.7-2.8	0.7-0.9	1.45-1.5
Copper			
Sands	2.6-2.8	0.9-1.4	1.1-1.45
Taconite			
Sands	3.0	0.7	1.75
Slimes	3.1	1.1	1.5
	3.1-3.3	0.9-1.2	1.55-1.7
Phosphate			
Slimes	2.5-2.8	11	0.25
Gypsum treated	2.4	0.7-1.5	
Bauxite			
Slimes	2.8-3.3	8	0.3 (2)

(1) For hard rock tailings; (2) Low by Australian standards 0.5-0.9 t/m^3 more likely.

higher densities correspond to greater depths within a deposit, and would be typical of tailings which have been placed into a water pond, i.e. sub-aqueous placement. If desiccation of the tailings occurs (i.e. drying in the sun), dry densities would usually be 20 to 50% higher than those shown.

The in-place density also depends on the thickness of tailings (the thicker the tailings the higher the average density), and whether the tailings drain vertically towards the foundation (vertical drainage yields higher effective stresses and higher densities).

General trends are better shown by the void ratio because the dry density is significantly affected by the specific gravity of the particles. Vick (1983) indicates that most high strength rock (and even low strength rock) tailings will have in-place void ratio of 0.6 to 0.9 for sand sized particles, and slimes 0.7 to 1.3. The exceptions are phosphatic clays, bauxite (alumina) and oil sands and slimes.

It will be noted that the dry densities are very much lower than the specific gravity of the parent rock. Since, in many mining operations only a very small percentage of the ore is removed as mineral in the milling process, it would not be unusual for the tailings volume to be 150 to 200% of the original volume of the mined ore.

The relative density of the sand sized tailings has an important influence on the potential for liquefaction. The relative density of spigotted tailings sands above water, ie. in the beach zone, can be expected to be in the range 30 to 50% (Vick 1983). Morgenstern (1988) indicates that relative densities of 50 to 65% can be expected for sands placed above water, and 35 to 40% when placed underwater. De Groot et al. (1988) show relative densities from 10 to 50% (mainly 25 to 40%) for sands placed below water.

17.2.3.5 *Permeability*

The permeability of mine tailings can be as high as 10^{-4} m/sec for clean sands, and as low as 10^{-9} m/sec for well consolidated fine clayey tailings. The permeability depends on tailings particle size, mineralogy, dry density (or void ratio), method of deposition and whether the tailings are saturated or partially saturated. Typical values are given in Table 17.5.

17.2.3.6 *Properties of water in tailings*

The water (or liquor) accompanying tailings often contains dissolved salts, heavy metals, and other residual chemicals from the mineralogical process.

The water may be highly acid or alkaline, have high salts content, a heavy metals content, arsenic, cyanide etc.; each may have important implications on the impact of the tailings disposal area on the environment. Some common problems are:

Gold tailings – cyanide,

Coal tailings – high salts content,

Table 17.5. Typical tailings permeability (adapted from Vick 1983).

Type	Average permeability (m/sec)
Clean coarse, or cycloned sands with less than 15% fines	10^{-4} to 10^{-5}
Peripheral-discharged beach sands with up to 30% fines	10^{-5} to 5×10^{-6}
Nonplastic or low-plasticity slimes	10^{-7} to 5×10^{-9}
High plasticity slimes	10^{-6} to 10^{-10}

Copper, lead-zinc, tin tailings – heavy metals, sulphides (which oxidise and lead to leaching of heavy metals and yield acid water),

Aluminium tailings – caustic soda (NaOH),

Uranium tailings – heavy metals, Radon.

Others may have quite acceptable water quality, e.g. some bauxite and iron ore washeries.

17.3 METHODS OF TAILINGS DISCHARGE AND WATER RECOVERY

17.3.1 *Tailings discharge*

Most tailings are pumped to the tailings disposal area as a high water content (e.g. 25 to 40% solids content) slurry. They are then discharged into the storage from a single or small number of discharge points, or from spigots as shown in Figures 17.6 and 17.7. Where a single fixed discharge point is used, it gives little flexibility in management of the tailings and is usually undesirable.

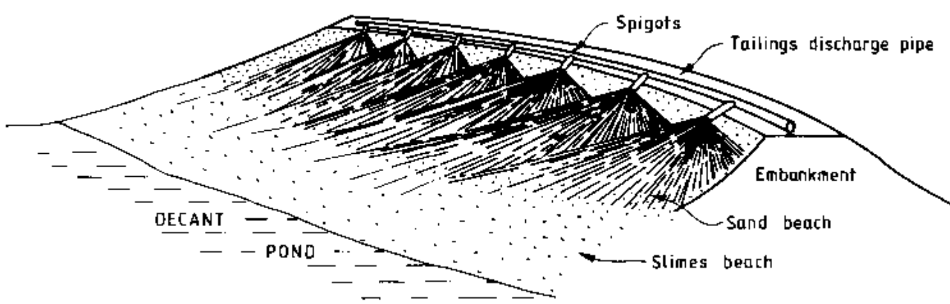


Figure 17.6. Tailings discharge by spigotting.

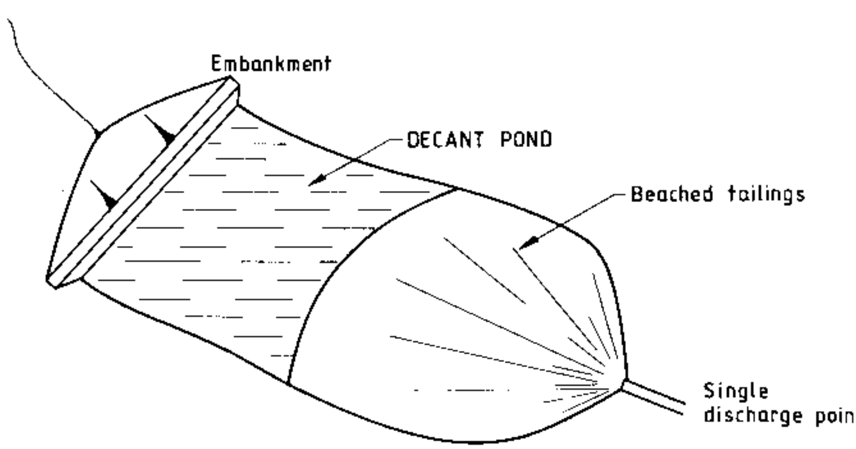


Figure 17.7. Tailings discharge from a single pipe.

For some fine grained high clay content tailings, a 'spray bar' is attached to each spigot to further distribute the tailings and allow uniform beaching. The 'spray bar' consists of, for example, a 100 mm diameter pipe with 20 mm diameter holes at 500 mm spacing. The idea is to give a uniform deposition of tailings, allowing the greatest degree of sorting (or 'classifying') of the tailings away from the discharge points, and optimising of desiccation.

It is usually preferable to discharge the tailings from the dam embankment, so that the tailings cover the storage area in the vicinity of the dam, and potentially reduce seepage. In the case of upstream or centreline type construction, it is essential that the tailings are discharged from the embankment and the water pond kept small.

17.3.2 *Cyclones*

When tailings dams are to be constructed by the upstream or centreline methods, it is common to separate the coarse fraction of the tailings from the fine fraction or 'slimes' by use of cyclones.

Figure 17.8 shows a typical cyclone. The 'whole' tailings are fed under pressure into the cylindrical cyclone. The coarser particles separate from the fines by centrifugal force (there are no moving parts), and spiral downward through the conical section as 'underflow.' The finer fractions and most of the water rise to the top outlet as 'overflow.'

The performance of a cyclone depends on many factors including the particle size, specific gravity, clay content of tailings, size of cyclone, pressure etc and advice should be sought from the cyclone manufacturer as to how effective they are likely to be.

The use of cyclones is discussed in Vick (1983). He indicates that reasonably clean sand can be obtained from tailings with less than 60% passing 75 μm, provided the tailings are essentially

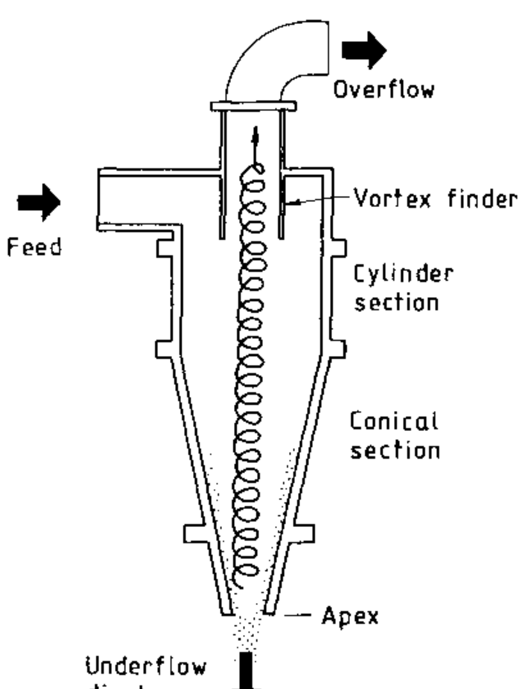


Figure 17.8. Typical cyclone for tailings.

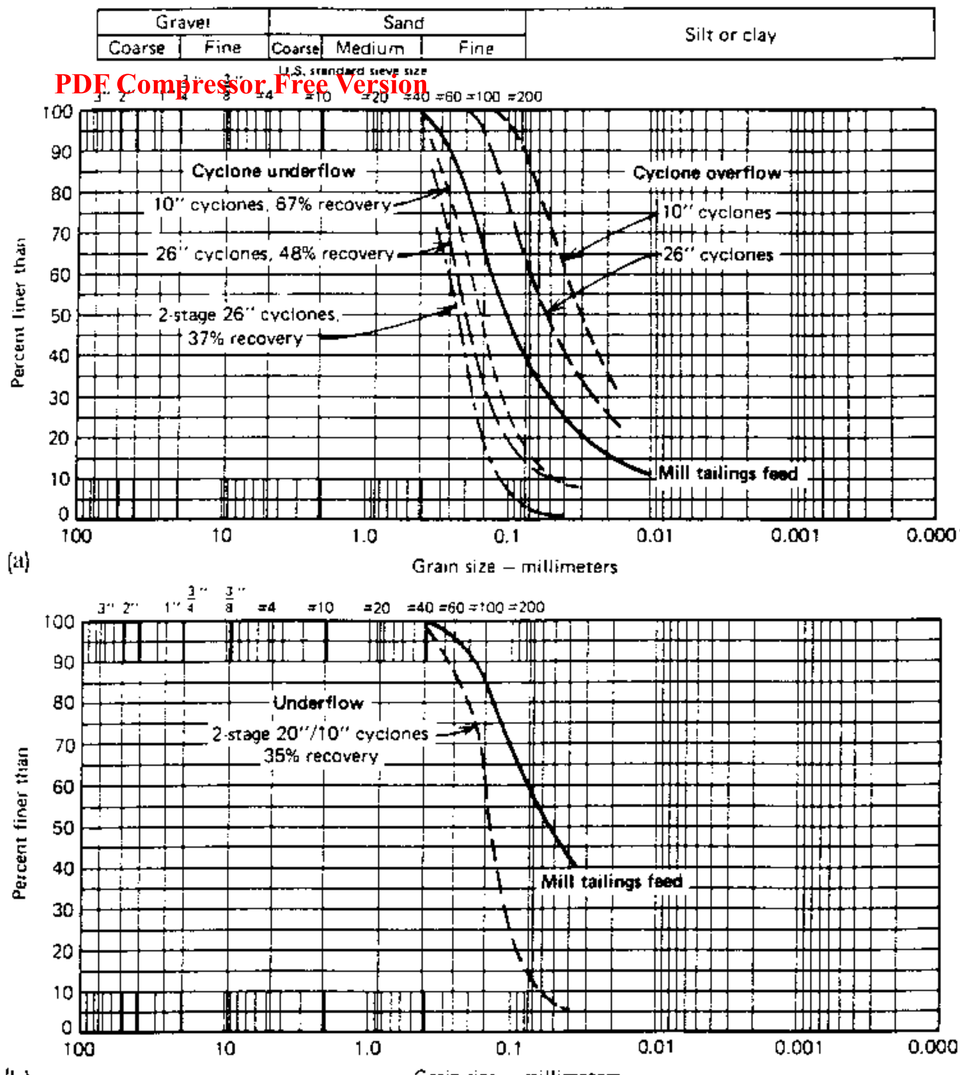


Figure 17.9. Results of cycloning tailings a) coarse feed, b) fine feed (Vick 1983).

non plastic and free of clay minerals. Such sands will have high permeability and high shear strength. Figure 17.9 shows some results of cycloning coarse and fine tailings.

Cycloning may be used in several ways:

– Stationary cyclone plant. A central high capacity cyclone station is established near the dam. Sands from the cyclone can be placed in the dam by conventional earthmoving equipment, and may be compacted. This may facilitate use of tailings for dam construction in seismic areas.

– On dam cycloning. The most common technique. Several small cyclones are set up on the dam and the underflow discharged onto the embankment. The cyclones are moved as the dam is raised. Vick (1983) indicates that relative densities of 45 to 55% are likely to be achieved if no compaction is carried out.

– Hydraulic cell method. The underflow is pumped from a single cyclone unit and placed in the dam in small 'paddocks,' and the water allowed to drain. This can result in high relative densities without compaction equipment.

17.3.3 *Subaqueous vs subaerial deposition*

If for climatic reasons, lack of a large discharge area or other reasons, tailings are deposited under water, the method is known as subaqueous deposition. When fine, high clay content tailings are deposited subaqueously, they will usually achieve a low settled density, very low shear strength and will be very compressible. Thus large, costly storages are required to contain the same quantity of tailings, and long term rehabilitation is more difficult than if subaerial deposition is used, because the surface of the tailings is wet and low strength. If water cover is maintained on the tailings, seepage quantities will be relatively high.

The method where the tailings are discharged in a 'beach' above the water pond, and where the deposition is cycled so that the tailings are allowed to dry by desiccation is known as subaerial deposition.

Desiccation induces negative pore pressures in the tailings, which results in consolidation of the tailings to higher densities than by subaqueous methods. The permeability and compressibility are also reduced.

In the ideal situation where the climate, storage area and tailings permit, the tailings will remain in a partially saturated condition, resulting in significantly reduced seepage from the storage. Post placement consolidation may be very small and the strength such that access onto the surface for rehabilitation can be easy. Figure 17.10 shows a conceptual design of such a system (Robertson et al. 1978).

Table 17.6 summarizes some of the advantages and disadvantages of subaerial depositions.

Blight (1988) also discusses the benefits and disadvantages of desiccation. He points out the risks of planning to use the benefits, and then not being able to achieve them. He cites an example where loss of control of water return resulted in the achieved dry density being about half the expected. The authors' experience is that the technique can be successfully used, if an adequate area is provided to accommodate the rate of tailings production, the evaporation is sufficiently high, and care is taken to operate the area properly. In such cases, even high clay content tailings have been desiccated sufficiently for use with upstream construction.

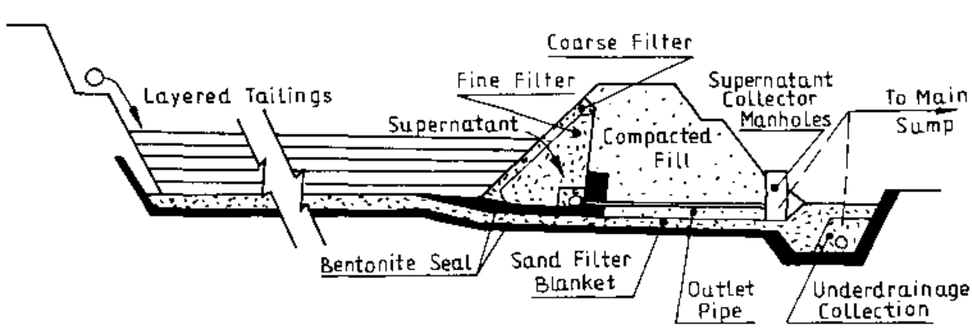


Figure 17.10. Subaerial placement of tailings (Robertson et al. 1978).

Table 17.6. Subaerial disposal advantages and disadvantages (Knight & Haile 1983; Lighthall 1987; from Ritcey 1989).

Advantages

a) Higher unit weights are achieved in the tailings mass, resulting in better utilization of the storage facility.
b) The drained nature of the tailings and removal of surface ponding adjacent to embankments, allows construction by upstream methods.
c) The low permeability, laminated structure of the tailings deposit, reduces seepage.
d) At decommissioning, the tailings are fully drained and consolidated, allowing immediate construction of a surface seal and cover and elimination of any long term seepage.
e) The drained nature of the tailings increases resistance to liquefaction.
f) There is a low hydraulic head on the liners and, therefore, a wide choice of liners are possible.
g) Capping and covering should be facilitated.

Disadvantages

a) Dusting may occur unless wetted.
b) Method may not be suitable during extreme cold periods. Therefore the disposal area is divided into two, for alternative summer-winter deposition. Winter deposition is, therefore, somewhat conventional (subaqueous).
c) Possibly higher initial cost to construct compared to other methods.
d) Requires a separate water storage reservoir which can be economically constructed for tailings water and runoff water recycling or discharge.

However, the authors have also seen cases where increased production of tailings, poor control of return water, low winter evaporation, and a lack of consideration of the fact that early in a dam's life the surface area is often less than later, when the tailings level has risen, have led to virtually no desiccation being achieved.

Blight (1988) also cautions against drying the tailings to the stage where they crack, as this may lead to a potential for piping failure. This would only be a problem in particular instances of upstream construction.

In general, one will not aim for the 'full' subaerial deposition method shown in Figure 17.10. This was developed for uranium tailings, where requirements for seepage control are very stringent. The costs of underdrainage may not be warranted in many cases. One should, however, aim to obtain as much benefit as possible from desiccation for the reasons outlined above.

17.3.4 *Water Recovery*

Water is removed from the tailings storage as it 'bleeds' from the beached tailings, or as tailings settle and consolidate if deposited subaqueously.

This is usually achieved by a pump mounted on a floating barge or a decant tower as shown in Figure 17.11.

There is no preferred method, each case having to be considered on its merits. It is usually desirable to keep the water surface area to a minimum so as to reduce evaporation of water (and hence reduce total water losses), and to optimise the benefits of desiccation of the beached tailings.

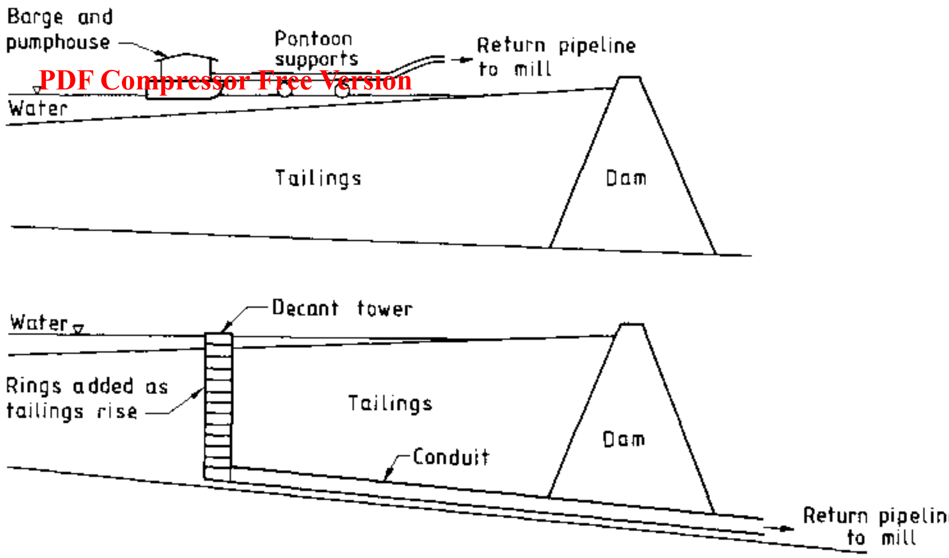


Figure 17.11. Water recovery methods.

17.4 PREDICTION OF TAILINGS PROPERTIES

17.4.1 *Beach slopes and slopes below water*

When tailings are deposited in a tailings dam they form a beach above the water level, and also fill the storage below the water level with a sloping surface on the tailings. The beach above the water level is commonly steeper adjacent to the discharge point than further away.

Figure 17.12 shows observed beach profiles for several dams containing platinum tailings (Blight 1988).

Figure 17.13 shows observed profiles for several tailings dams, in which fine grained tailings with high clay contents are stored. Properties for these tailings are given in Figure 17.5 and Tables 17.2 and 17.3.

The fact that the tailings are deposited in this way is important because it results in a reduction in the available tailings storage volume compared to water storage, and it determines the position of the water pond, which is important, particularly for upstream construction.

Blight and his co-workers have investigated beach profiles by laboratory experiments in sloping flumes, and by observing beach profiles in the field. This work is described in Blight, Thomson & Vorster (1985), Blight & Bentel (1983) and Blight (1987,1988). The concept they have developed was originally proposed by Mellent et al. (1973). It was found that by plotting dimensionless parameters H/Y versus H/X, as shown in Figure 17.14, the different profiles in Figure 17.12 all plotted to give a single 'master' beach profile with the equation

$$h/Y = (Y/X)(1 - H/X)^n = i_{av}(1 - H/X)^n$$

The master profile is different for each type of tailings. The particle size, specific gravity and solids content of the tailings all affect the profile. Blight (1988) indicates that the master profile applies, almost, regardless of the length of the beach, and hence laboratory flume tests can be used to predict the behaviour of the dam.

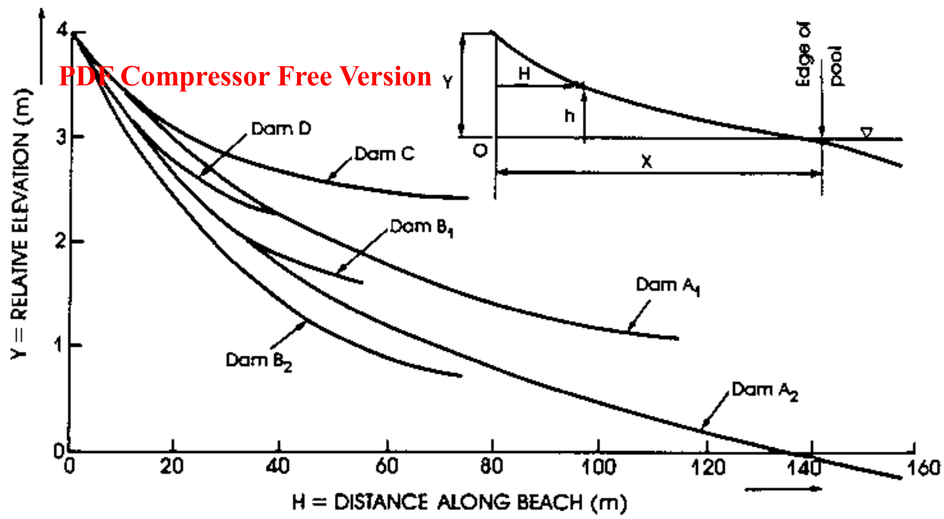


Figure 17.12. Measured beach profiles on six platinum tailings dams (Blight 1988).

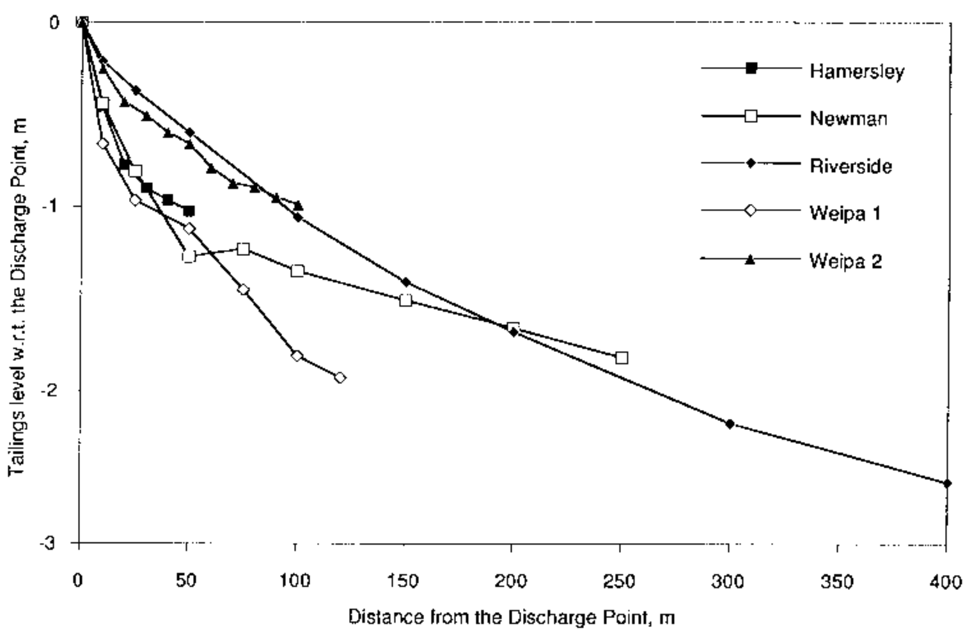


Figure 17.13. Measured beach profiles, fine grained high clay content tailings.

Morgenstern & Kupper (1988) indicate that, in their experience, the laboratory flume tests did not show consistent trends with field behaviour. Fourie (1988) describes the use of such flume experiements on some fine grained bauxite, ñickel and coal washery tailings. These gave the rather unusual result in that the dimensionless beach profile is concave downwards – quite

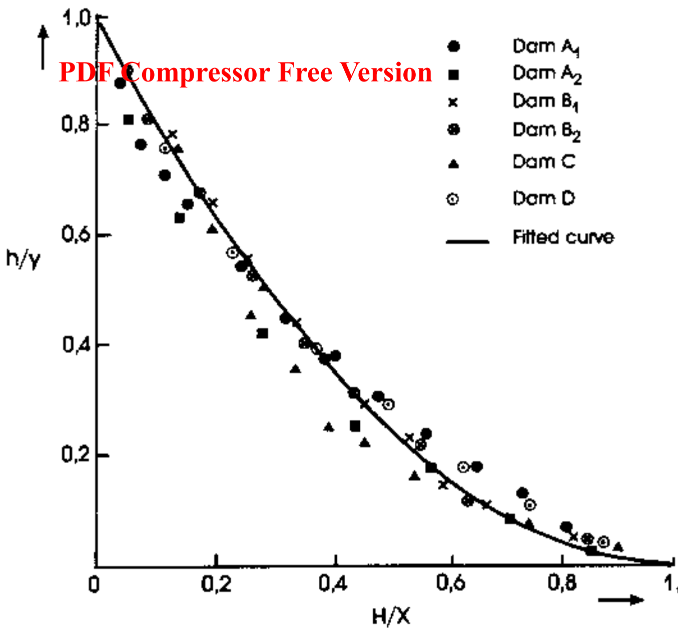


Figure 17.14. Dimensionless beach profile for platinum tailings in Figure 17.12 (Blight 1988).

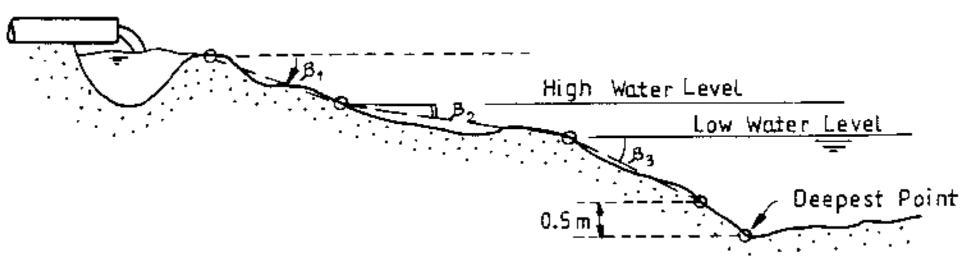


Figure 17.15. Beach profile definitions (de Groot et al. 1988).

contrary to observed field behaviour. As shown in Figure 17.13, the authors' observation is that high clay content tailings do beach concave upwards in the field, with the sand fraction settling out close to the discharge point. Based on this, and Fourie's tests, it would seem wise to be cautious in using flume tests for high clay content tailings.

The concept of a master profile is, however, well established and has been noted by others, e.g. Smith et al. (1986). Hence, one can predict later performance quite well from the initial behaviour of a tailings storage.

However, for planning purposes in a new project, one will have to rely on flume tests, and relationship to other measured profiles.

De Groot et al. (1988) present some interesting theoretical laboratory and field observation data relating to the beach angles of dredged sand. They consider slopes above and below water level, and in the tidal zone (not generally present in tailings dams). Figure 17.15 shows the slope profile definitions.

They concluded that the beach slope above water (β_1) could be predicted from

$$\text{Tan } \beta_1 = 0.006 \left(\frac{D}{D_o} - 1\right) \left(\frac{q}{q_o}\right)^{-0.45}$$

where $D = D_{50}$ size of sand (μm)

q = specific mixture flow rate = flow rate/m width (m^3/m/sec)

D_o = 65 μm

q_o = 1 m^3/m/sec.

This equation only applies for $D > D_o$ and $q > 0.01\ q_o$ so will not apply to finer fractions of tailings. It does show that the slope angle is dependent not only on particle size, but also on discharge rate, with steeper slopes for lower discharge rates/m width. It would appear to have application to tailings in the vicinity of the discharge point, where the coarse fraction is settling out. However, the authors have no experience of doing this so caution must be advised.

De Groot et al. (1988) also present information relating to observed beach slope angles below water (β_3). Results are shown in Figure 17.16.

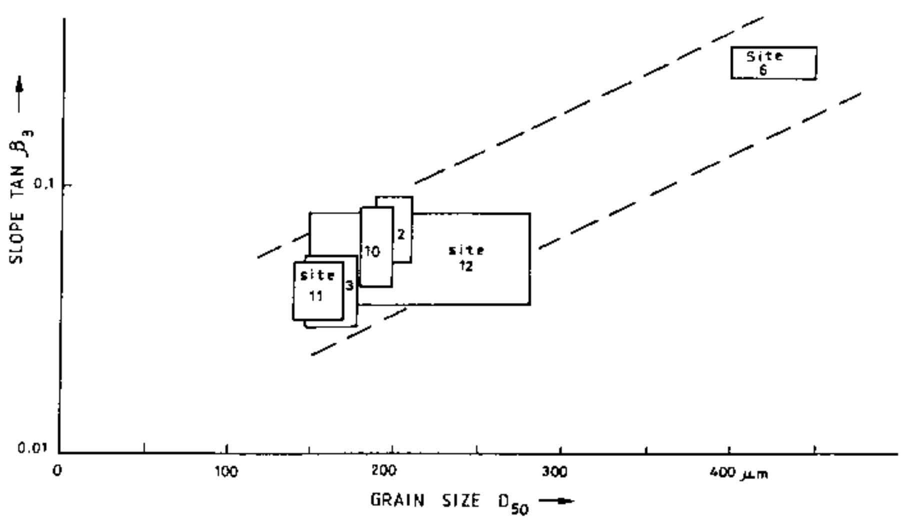


Figure 17.16. Observed slopes below water level (after de Groot et al. 1988).

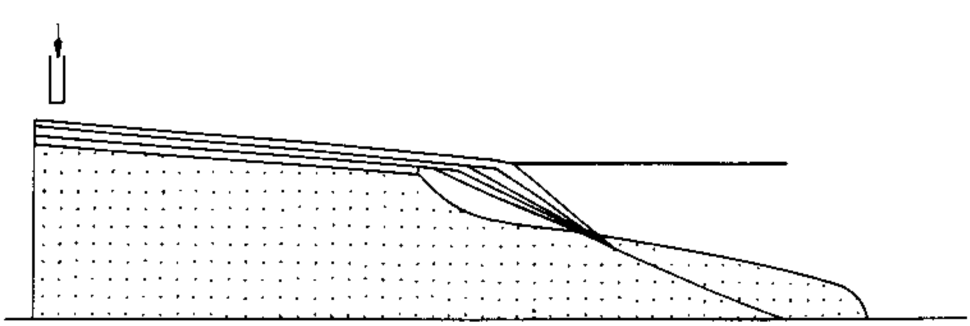


Figure 17.17. Flow slide mechanism leading to flat slopes below water level (de Groot et al. 1988).

They point out that the observed slopes below water are much flatter than the predicted equilibrium slope (which might be expected to be steeper than that above water because of reduced gravitational forces below water). They explain this as being due to flow slides (see Fig. 17.17) where the upper part of a slope oversteepens as sedimentation occurs, results in slope instability, and a flow which produces much flatter slopes. They note that observed slopes below water were similar to that in the beach above water. This is also the authors' observation.

17.4.2 *Particle sorting*

Because the coarser particles contained in the tailings settle more rapidly than the finer particles, a gradation of particle size occurs on the beach, with coarser particles depositing near the discharge point. Particle sorting also occurs within a layer, with coarse, high specific gravity particles settling to the base of a layer. These can form a sand 'parting' even in high fines content tailings. Conversely, if the coarse particles are low specific gravity as occurring in coal washery tailings, they will 'float' to the surface of a layer.

The lateral variations in grading with distances from the discharge point can have an important effect on tailings permeability – with the coarser more permeable tailings deposited near the discharge spigots, and the finer low permeability slimes deposited further away. This is used in the upstream method of tailings dam construction to control seepage pore pressures and, hence, to maintain stability. Blight & Bentel (1983) and Blight (1987, 1988) discuss this feature and conclude that the particle size distribution can be roughly predicted from

$$A = e^{-BH/X}$$

where $A = \dfrac{D_{50}\ \text{(at distance H down the beach)}}{D_{50}\ \text{(of the total tailings)}}$

B = characteristic of tailings and is a function of the discharge rate, and the specific gravity of the tailings

X = length of beach as in Figure 17.14.

Abadjiev (1985) presents a similar formula which can be applied to not only the D_{50} size but to D_{90}, D_{60}, D_{10} etc. He also presents data from several tailings dams showing the lateral gradation.

The authors' experience is that the amount of sorting which occurs is dependent on the method of deposition. Spray bars, and closely spaced spigots, give very good sorting provided discharge rates are kept low. This is consistent with the observations of Blight and Abadjiev.

17.4.3 *Permeability*

The permeability of tailings is dependent on the particle size distribution and void ratio, so is, therefore, dependent on the distance from the discharge point, the method of deposition, e.g. subaqueous or subaerial deposition, and the depth in the storage.

The following procedure is, therefore, recommended for the estimation of permeability at any particular place: (1) Predict the particle size at that place from the grading of the whole tailings, and the sorting as predicted by the approaches discussed above in Section 17.4.2. (2) Predict the void ratio from sedimentation, and the sorting as predicted by the approaches discussed above in Section 17.4.2. (3) Prepare samples based on (1) and (2). (4) Calculate permeability from either falling head or constant head tests, or back calculation from consolidation tests, conducted on these samples.

For sand sized tailings the permeability can be approximately estimated from Hazen's formula, i.e.

$$k = D_{10}^2 / 100$$

where k = permeability in m/sec
D_{10} = grain size in millimetres, for which 10% of the particles are finer.
Using this, and the relationship between particle size and distance from the discharge point discussed in Section 17.4.2, Blight (1987) suggests that the permeability of the beached tailings will vary as:

$$k = ae^{-bH}$$

where a and b are dependent on the tailings
H = the distance down the beach.

17.4.4 *Dry density*

Prediction of the dry density of tailings in the storage is of prime importance, because the amount of tailings which can be stored in a given storage volume depends on the density achieved. This, in turn, is dependent on the type of tailings, method of deposition (subaqueous or subaerial), drainage conditions (e.g. if underdrains are provided or if the soil and rock underlying a tailings dam is of high permeability, higher effective stresses result in higher densities), degree of desiccation, distance from the discharge point and proximity to the water pond etc.

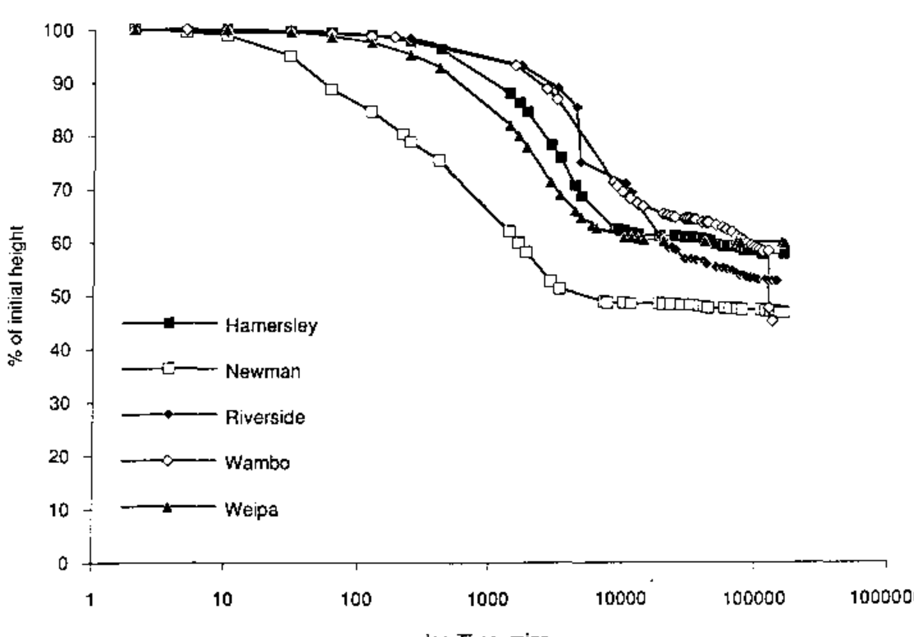


Figure 17.18. Settling tests on mine tailings.

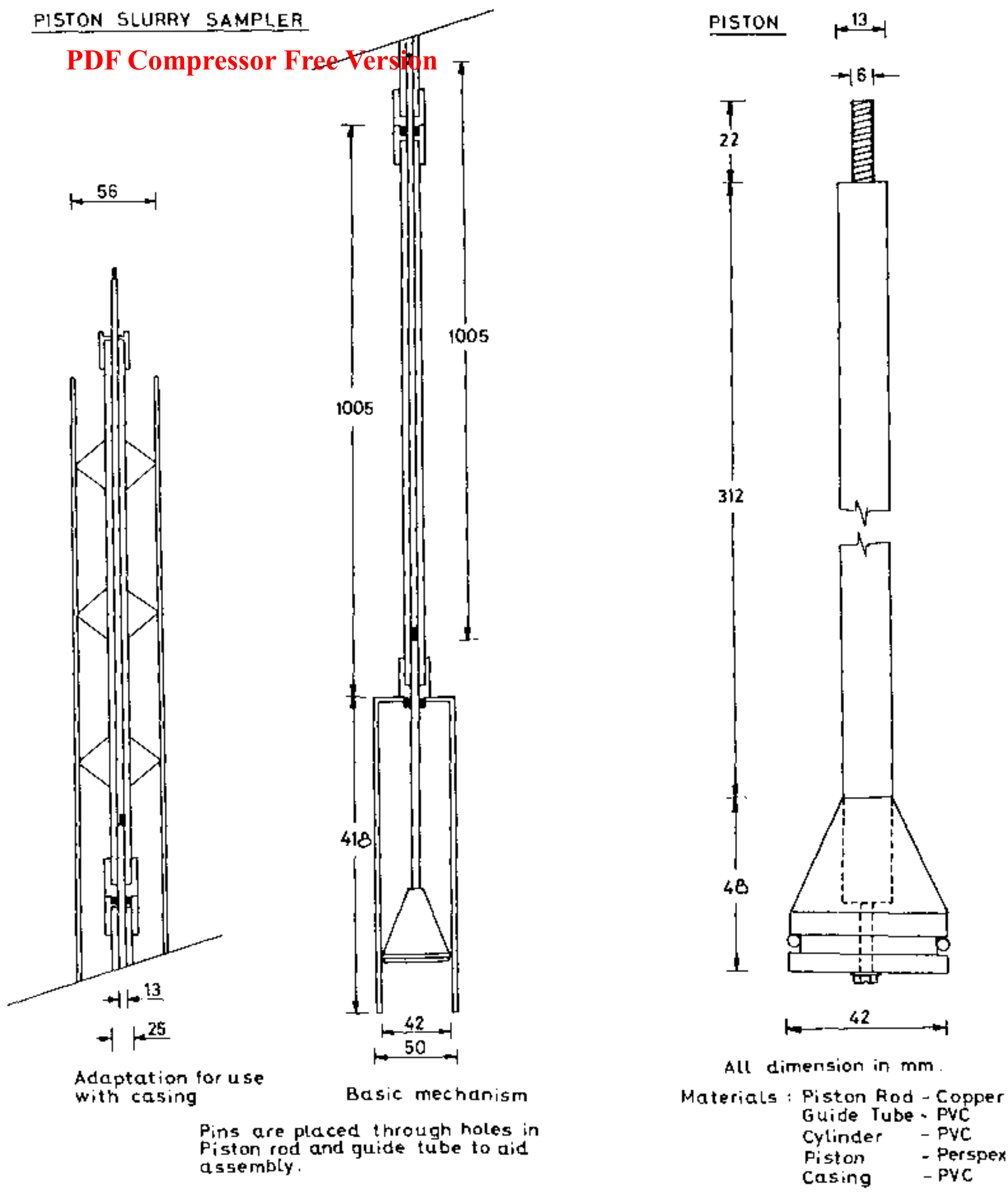


Figure 17.19. The Hogg sampler for very soft and loose tailings.

For tailings which are deposited under water there are two phases:

a) sedimentation

b) consolidation under self weight and the weight of tailings above.

The sedimentation phase occurs when the tailings are first placed in the dam, and the particles settle, with a clear water interface forming on the surface of the tailings. In subaerial deposition, this 'bleed' water will run off down the beach slope.

The consolidation phase follows, and may occur concurrently with sedimentation. Prediction of consolidation of tailings requires a different approach to that for normal soils, because of the large strains involved. The traditional one dimensional Terzaghi theory is not applicable, often resulting in overestimation of the time of consolidation and underestimating pore pressures. It is also necessary to use special slurry consolidometers, so tests can begin at the density of the tailings after sedimentation.

Schiffman & Carrier (1990) and Schiffman et al. (1988) give overviews of this topic, and reference other papers which can be used to obtain the detail of the testing and analysis techniques. They indicate that centrifuge testing can also be used for more accurate modelling, but advocate monitoring and analysis of the behaviour of the early phases of tailings deposition as the best means of predicting behaviour.

The reader is referred to these references rather than repeat the information here. However, the following practical points are based on the authors' experience:

– It is essential to carry out testing with water which is representative of the actual tailings water. It is also essential that the tailings have not dried out before carrying out testing, particularly if the tailings have a significant clay content. Drying completely changes the sedimentation behaviour (usually makes the tailings settle more quickly and more densely), and affects the consolidation and permeability properties.

– Sedimentation tests, where the tailings are mixed and placed in a 500 mm column and allowed to settle, gives a good indication of the minimum settled density. Figure 17.18 gives the results of such tests for the tailings in Tables 17.2 and 17.3 in 500 mm columns. Field measurements indicate that the 500 mm columns slightly underestimate the field density of tailings.

– Field sampling of the very loose/soft upper layer of tailings is very difficult. Ritcey (1989) refers to the use of piston samplers. A simple device which was developed by R. Hogg at the University of New South Wales, and has been used to obtain disturbed samples of tailings, is shown in Figure 17.19.

The disturbed samples allow determination of water content and hence dry density, and give samples for particle size distribution testing.

17.4.5 *The prediction of desiccation rates*

If one is to rely on the increased dry density and strength which can be obtained from desiccation of the tailings, it is necessary to predict the rate of drying. The rate is dependent on:

– the properties of the tailings, in particular the settled water content, the permeability and the suction pressure which develops on drying

– the climate; evaporation and rainfall

– the deposition cycle; depth of and time between each cycle of deposition.

In Swarbrick & Fell (1990, 1991) the results of a research program to develop a method for predicting desiccation rates is described. Based on laboratory and field drying experiments, it has been shown that desiccation occurs as:

a) Settling, until the rate of water release equals the potential evaporation

b) Stage 1 drying, which occurs at a linear rate with time, generally at the same rate as from a free water surface

c) Stage 2 drying, which occurs at a decreasing rate. This decreasing rate has been shown to satisfy the sorptivity equation i.e.

$$E_{cum} = b\sqrt{t}$$

where E_{cum} = cumulative evaporation after the linear stage
  t = time after the linear stage – days

  b = sorptivity coefficient mm days $^{-0.5}$.

17.4.6 *Drained and undrained shear strength*

17.4.6.1 *Drained shear strength*

The effective strength parameters c′, ϕ′ for tailings can be obtained from triaxial or direct shear tests on samples of the tailings. It is particularly important that sandy tailings are tested at the correct relative density and stress range, because of the strong dependence of ϕ′ on the relative density, and the curved nature of the Mohr envelope.

Table 17.7 shows some typical values of effective friction angle. It will be noted that most are relatively high, reflecting the absence of any structure and the grinding of rock to angular particles. Unless the tailings cement on deposition, e.g. gypsum, the effective cohesion is c′ = 0.

17.4.6.2 *Undrained shear strength*

The undrained shear strength of the 'slimes' or clay-silt sized part of the tailings can be important both from the view of overall slope stability in upstream construction, and the strength of the surface of the tailings for rehabilitation.

The strength can be determined by appropriate triaxial testing, e.g. using the SHANSEP method (Ladd & Foott 1974) but must be related to the field situation by a knowledge of the degree of overconsolidation of the tailings. This can be done by taking undisturbed samples of the tailings and carrying out oedometer consolidation tests.

The undrained strength of tailings in-situ may be measured using a vane shear. However, the results are often very scattered because of the varying degrees of overconsolidation within layers which have been partly desiccated, and the presence of sandy layers which lead to overestimation of undrained strength. Figure 17.20 shows some results from the Newman

Table 17.7. Typical values of effective friction angle ϕ′ of tailings (adapted from Vick 1983).

Material	ϕ′ (degrees)	Effective stress range kPA
Copper		
Sands	34	750
	33-37	625
Slimes	33-37	625
Molybdenum beach sands	33-38	
Taconite		
Sands	34.5-36.5	–
Slimes	33.5-35	–
Lead-zinc-silver		
Sands	33.5-35	–
Slimes	30-36	–
Gold slimes	28-40.5	900
Fine coal refuse	22-39	270
	22-35	1100
Bauxite slimes	42	175
Gypsum tailings	32 (c' = 22 kPa)	450

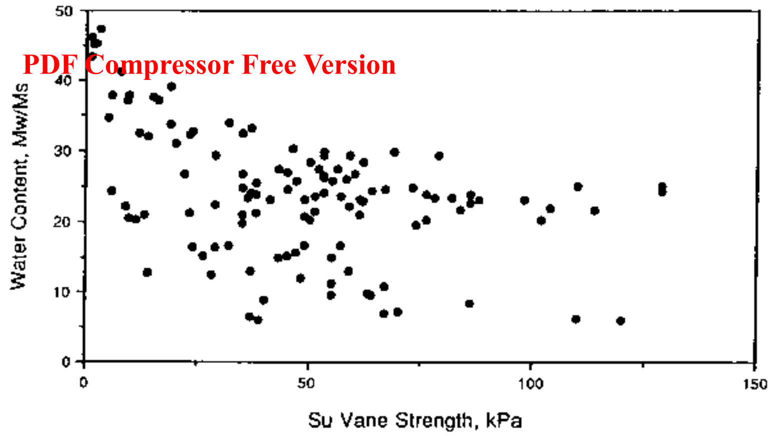


Figure 17.20. Vane shear test results, Newman tailings.

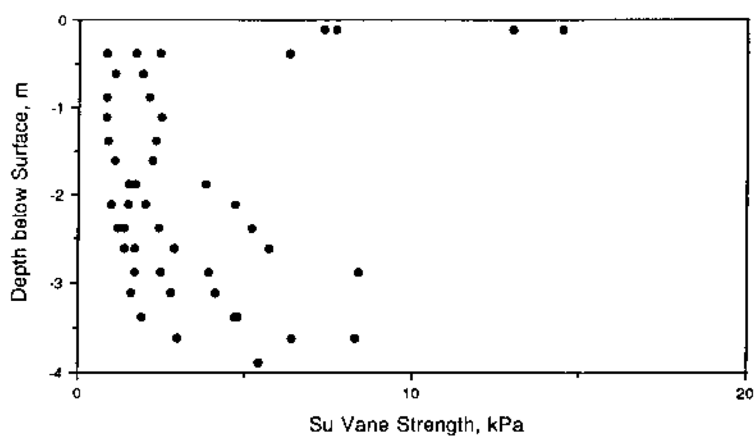


Figure 17.21. Vane shear test results, Wambo tailings.

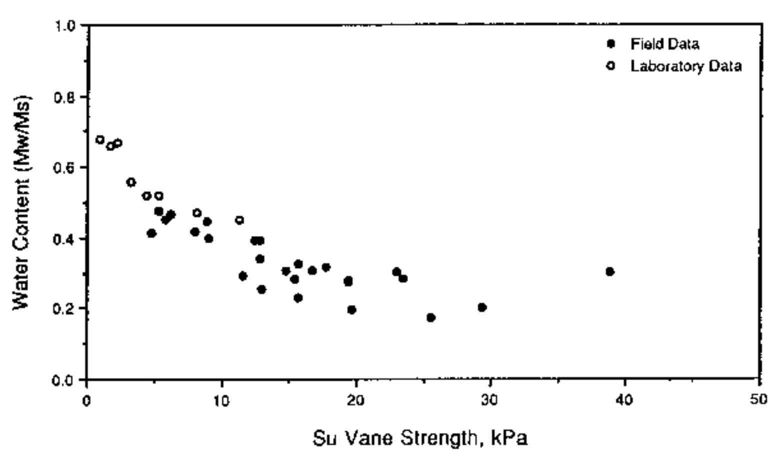


Figure 17.22. Undrained strength vs water content, Weipa tailings.

tailings, which show the wide scatter and lack of relationship to water content.

Where tailings are consistently fine grained, e.g. well away from the discharge point, and not affected by desiccation other than in a single drying phase, reasonably constant strengths can be obtained by a vane shear. Figure 17.21 shows such a case with a clearly defined overconsolidated 'crust' on the surface, due to desiccation.

The authors (Swarbrick & Fell 1991) have used vane strengths in the laboratory to develop a relationship between undrained strength and water content. Here some consistency was possible, with good comparison with single stage desiccation in the field. The laboratory tests were done using a miniature vane, with samples dried in glass beakers. Figure 17.22 shows the results.

17.5 METHODS OF CONSTRUCTION OF TAILINGS 'DAMS'

17.5.1 *General*

In most cases the basic requirements for a tailings disposal area are to store the tailings in such a way that they remain stable (i.e. there are no slope stability problems), and do not impact excessively on the environment by water pollution, and/or wind and water erosion. Only in some cases is it necessary, or desirable, to store water in the tailings disposal area.

This should be kept in mind when designing a tailings embankment, and one should not always feel it necessary to construct 'a dam' to store the tailings.

Many of the principles and practices developed in conventional water dam engineering are applicable to tailings, but it should not be assumed that all water dam technology or philosophy is applicable.

In many cases it is better to consider tailings embankments as landfills, or dumps of fine grained soil which have to be engineered to:

– optimise the amount of tailings which can be stored in a particular area,

– control piezometric conditions to ensure adequate strength and stability,

– control the environmental impact.

Australian practice has in the past been (and to a large extent still is) biased towards constructing conventional dams to store tailings. This approach also applies in some other countries. This is possibly due to:

– many designers having a background in water dam engineering,

– regulatory authorities requiring such an approach (largely because of their background in water dam engineering),

– a relatively wet climate in many mining areas, and many mines having very fine grained and/or oxidised tailings which are not readily utilized in 'upstream' construction methods.

There is a recognition amongst many practitioners that this philosophy is costly and unnecessary, and alternative methods are being used.

The following discussion is intended to give an overview of the various methods and their limitations, with a view to encouraging consideration of the alternatives, rather than accepting conventional water dam technology.

17.5.2 *Construction using tailings*

There are three main methods for constructing tailings embankments using the tailings as a

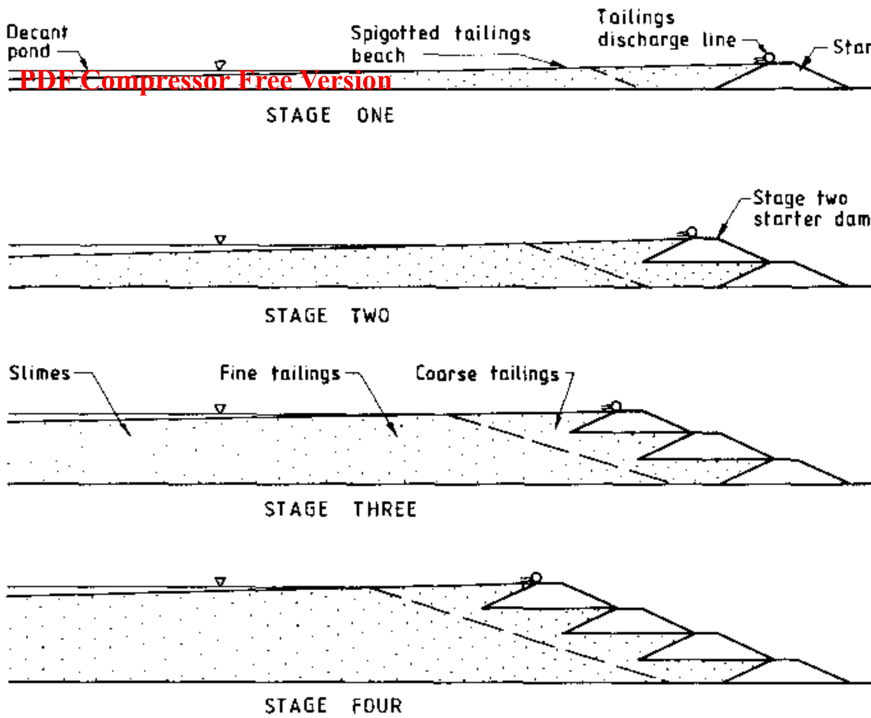


Figure 17.23. Construction of a tailings embankment using the upstream method.

major construction material:
- upstream method,
- downstream method,
- centreline method.

These are described in some detail in ICOLD (1982) and Vick (1983). Figure 17.23 shows the principles of the upstream method.

17.5.2.1 *Upstream method*

The important features of this approach are:

– The starter dam is essentially a containing embankment and a support for the tailings discharge line, rather than a dam in itself. The starter dam is best constructed of permeable rockfill (e.g. mine waste) to allow drainage of seepage water and to control erosion. However, it may be constructed of relatively impermeable rockfill, earthfill or even dried tailings pushed up by bulldozers from the tailings beach

– Tailings discharge must be controlled by, say, spigotting to ensure that the coarser sandy tailings are deposited near the starter dam. This is essential to control seepage pressures as outlined below.

– The water pond (decant pond) must be kept well away from the edge of the storage. If allowed to come close to the edge, the piezometric pressures will be high and slope instability may result.

– The coarse tailings may be at a low relative density and, if saturated and subject to

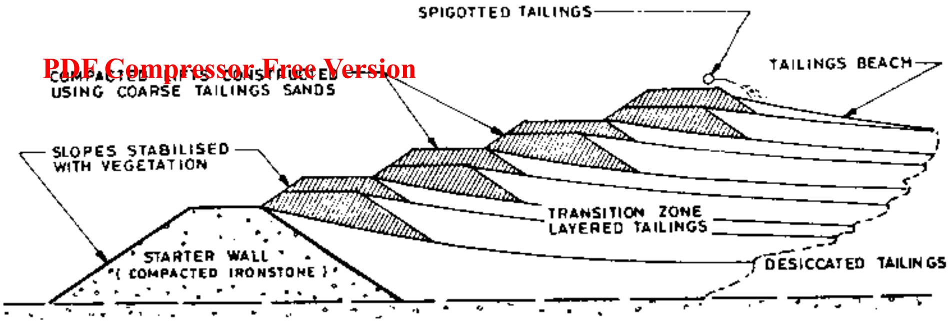


Figure 17.24. Section of bauxite tailings embankment at Weipa showing upstream construction (Minns 1988).

earthquake, may liquefy, leading to slope failure. Because of this the method is seldom used in high seismic risk areas.

The method is best suited to hard rock tailings, i.e. silt-sand tailings which are readily spigotted to classify into a sandy beach; but can be applied to high clay content tailings. Figure 17.24 shows use of the upstream method for bauxite tailings at Weipa (Weipa 1 tailings in Fig. 17.5).

A similar approach is being used for iron ore tailings at Newman. In both of these cases, stability will also be dependent on allowing the finer tailings ('slimes') to desiccate and develop a significant dried strength.

17.5.2.2 *Downstream method*

Figures 17.25 and 17.26 show two versions of the downstream method of tailings embankment construction.

The important features of this approach are:

– The embankment is constructed of selected material. In the case of Figure 17.25, a water storage dam type cross section has been used, with an upstream impervious zone and internal drainage control. This allows water to be stored adjacent to the embankment. In Figure 17.26 the embankment is constructed from the coarse underflow part of cycloned tailings. These may be compacted as described in Section 17.3.2. In this case, the stability is dependent on the water pond being kept well away from the embankment to control the position of the phreatic surface.

– The embankment is best constructed at least, partly, of permeable material to allow control of piezometric pressures.

– It is desirable, but not necessary, to place the tailings in a controlled manner. For Figure 17.25 type construction no control is necessary (from a stability viewpoint). For Figure 17.26 type construction, it would be important to spigot or cyclone the tailings to form a relatively sandy beach adjacent the downstream zones, so as to control piezometric pressures.

If rockfill rather than tailings underflow is used for the downstream zones in Figure 17.25, a graded filter zone may be required between the tailings and the rockfill to prevent erosion of the tailings into the rockfill.

A combination of upstream and downstream methods may be used. An example is shown in Figure 17.27.

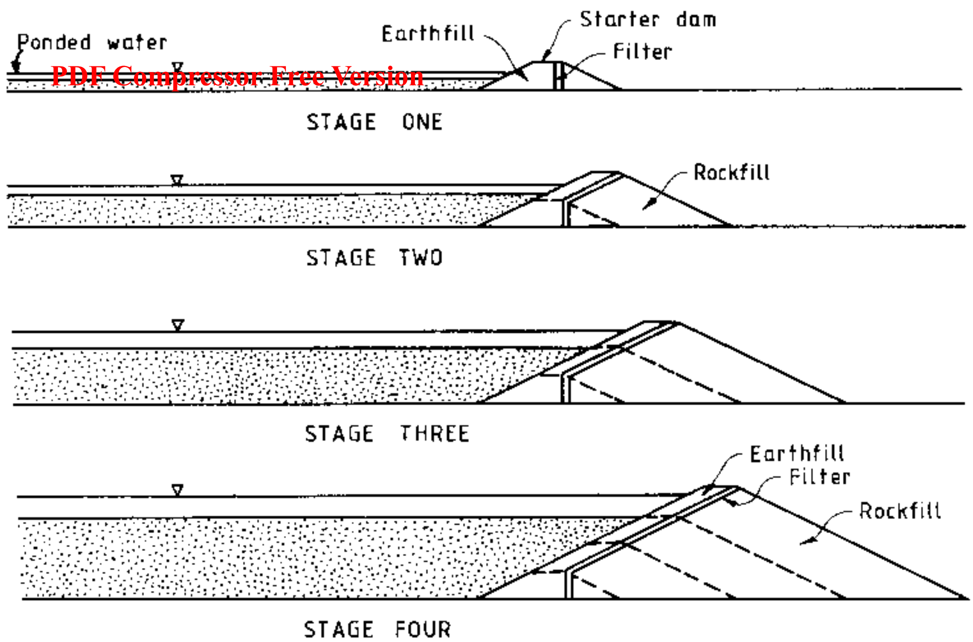


Figure 17.25. Tailings embankment construction using the downstream method and zoned earth and rockfill construction.

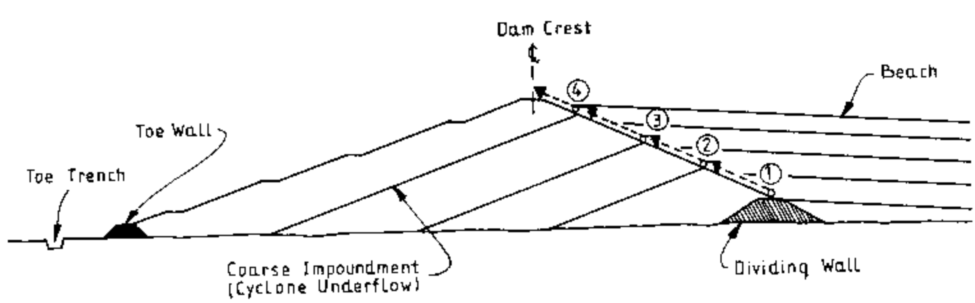


Figure 17.26. Tailings embankment construction using the downstream method and cycloned tailings (ICOLD 1982).

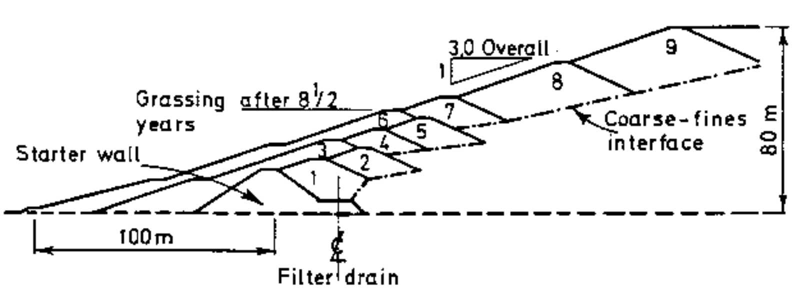


Figure 17.27. Combined upstream and downstream construction (Lyell & Prakke 1988).

17.5.2.3 *Centreline method*

Figures 17.28 and 17.29 show two versions of construction by the centreline method.

The important features of this approach are:

– The embankment is partly constructed of selected material. In Figure 17.28 a zoned water storage type section has been used for the downstream part of the embankment, with the upstream part constructed on spigotted tailings. In Figure 17.29 cycloned tailings have been used, with the cyclone underflow discharged on the downstream side and the overflow finer portion on the upstream side.

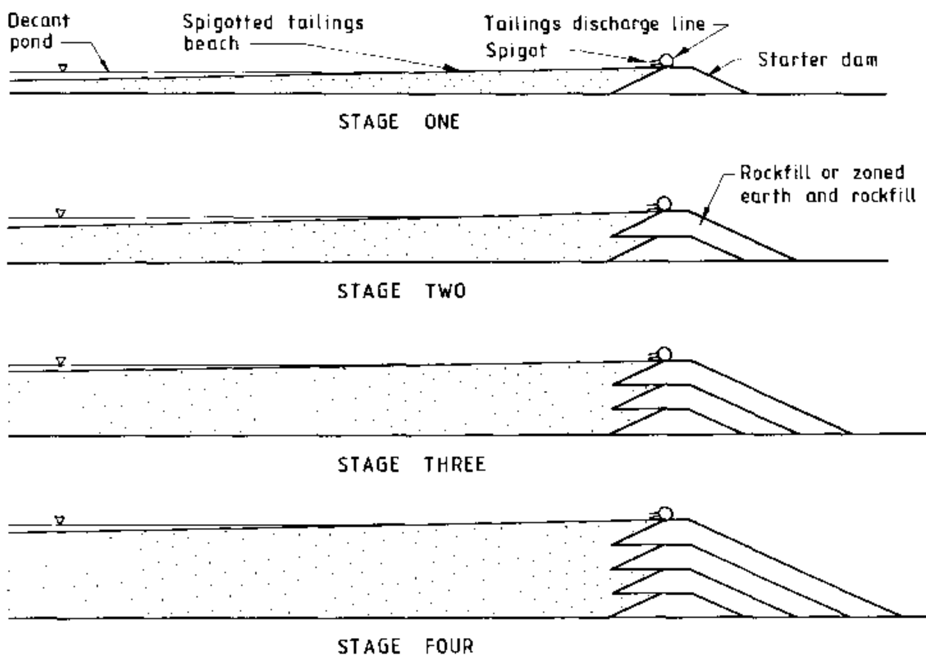


Figure 17.28. Construction of tailings embankment using the centreline method.

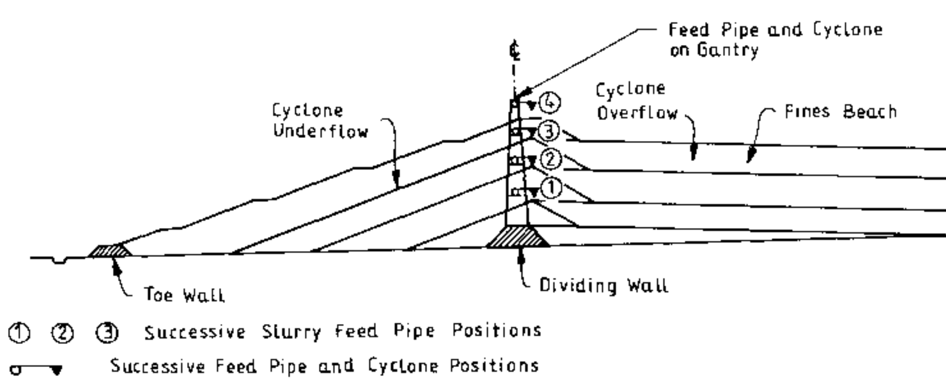


Figure 17.29. Construction of tailings embankment using the centreline method and cycloned tailings (adapted from ICOLD 1982).

– The embankment must be constructed at least, in part, of permeable material to allow control of piezometric pressure.

– The water pond must be kept away from the edge of the embankment to prevent buildup of excessive piezometric pressures.

– It is essential that tailings are placed in a controlled manner, i.e. spigotting for Figure 17.28, cycloning for Figure 17.29.

17.5.3 *Construction using conventional water dams*

Conventional earthfill, earth and rockfill, and faced rockfill dams are often used to store tailings.

Some factors which should be considered as variations on conventional water dam technology are:

– The tailings seepage water often has high salts content, which suppresses the likelihood of dispersion of clays, and allows use of less rigidly controlled filters. Stapledon et al. (1978) gives an example. This may allow use of a coarser reject from the milling operation as a filter, rather than using a processed sand/gravel, with resultant significant cost savings

– Leakage of water from the dam may not be critical, allowing use of higher permeability core material and not requiring foundation grouting. If the water is contaminated, seepage collection downstream may be necessary

– There is often a large quantity of waste rock available from the mining operation at a low cost. In these cases it may be better to use slightly flatter slopes, and not compact the rock with construction of the dam being done by mining equipment. If a well controlled earthfill zone with filters is required (i.e. sloping upstream core construction), this would usually be built by earthworks contractors after the rockfill with well controlled construction. The rockfill may have to be compacted to limit settlements in this situation

– Erosion of the upstream slope is seldom a problem for tailings dams. At most, waste rock should be dumped on the upstream face. Steeper upstream slopes can be used if the slope is supported by tailings and not subject to drawdown

– Staged construction is usually required, favouring sloping upstream core type cross sections.

Figure 17.30 shows an example of staged construction which minimises the volume of fill in each stage.

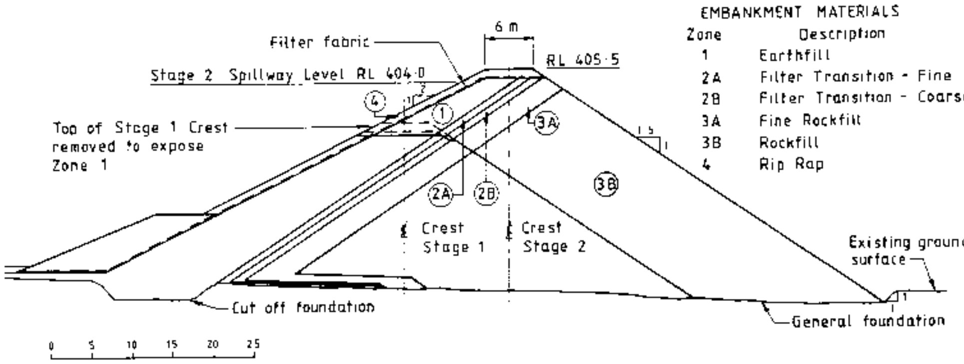


Figure 17.30. Staged construction, proposed Ben Lomond tailings dam (Coffey & Partners 1982).

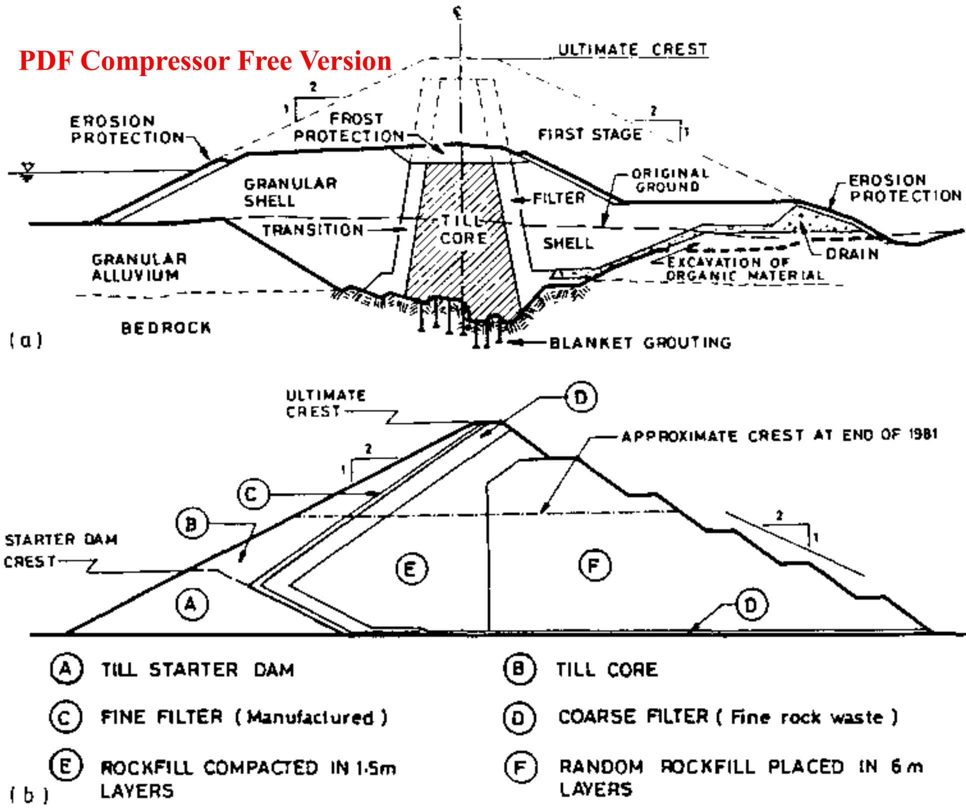


Figure 17.31. Staged construction of tailings dams a) Page-Williams Mine (Welch et al. 1987); b) Afton tailings dam (Lighthall 1987) from Minns (1988).

Figure 17.31 shows staged construction where because of the particular circumstances, different approaches to staging have been adopted.

17.5.4 *Selection of embankment construction method*

The method to be used will depend on the particular circumstances at the mine, e.g. type of tailings, production rate, climate, local topography, seismicity, availability of waste rock, regulatory authority requirements etc.

All methods allow staged construction of the embankment. This is the significant difference between water storage dams and tailings embankments, and allows minimisation of up-front capital works, and improvement of overall economies.

Table 17.8, which is taken from Vick (1983), compares the different methods of embankment construction. While the authors generally agree with the table, it is pointed out that the upstream method can be used for all types of tailings if they are allowed to desiccate properly.

One important factor in selection of embankment type is the degree of seepage control achievable. This is vital, because it determines the stability of the embankment and the steepness of the downstream slope. Figure 17.32 shows that downstream construction allows best

Table 17.8. Comparison of tailings embankment types (Vick 1983).

Embankment type	Mill tailings requirements	Discharge requirements	Water storage suitability	Seismic resistance	Raising rate restrictions	Embankment fill requirements	Relative embankment cost
(1) Water retention	Suitable for any type of tailings	Any discharge procedure suitable	Good	Good	Entire embankment constructed initially	Natural soil borrow	High
(2) Upstream	At least 40-60% sand in whole tailings. Low pulp density desirable to promote grain-size segregation	Peripheral discharge and well-controlled beach necessary	Not suitable for significant water storage	Poor in high seismic areas	Less than 5-10 m/yr most desirable. Greater than 17 m/yr can be hazardous	Natural soil, sand tailings, or mine waste	Low
(3) Downstream	Suitable for any type of tailings	Varies according to design details	Good	Good	None	Sand tailings or mine waste if production rates are sufficient, or natural soil	High
(4) Centerline	Sands or low-plasticity slimes	Peripheral discharge of at least nominal beach necessary	Not recommended for permanent storage. Temporary flood storage acceptable with proper design details	Acceptable	Height restrictions for individual raises may apply	Sand tailings or mine waste if production rates are sufficient, or natural soil	Moderate

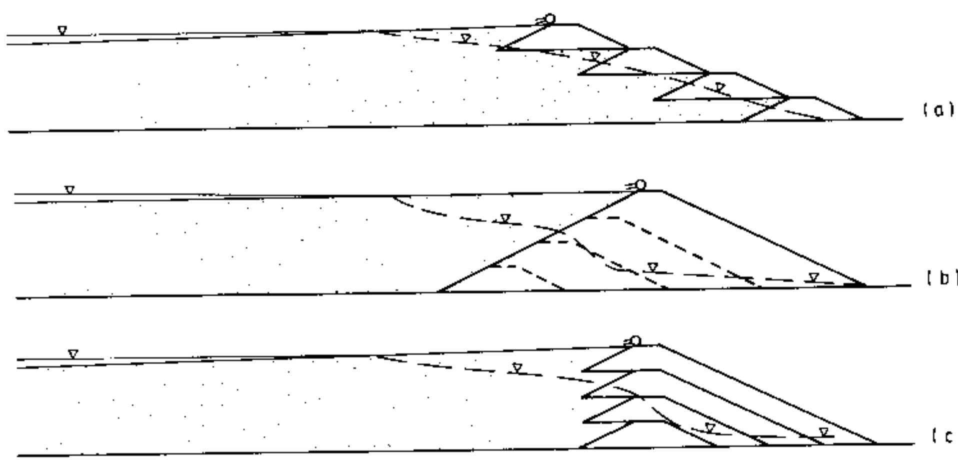


Figure 17.32. Internal seepage a) upstream embankment; b) downstream embankment; c) centreline embankment.

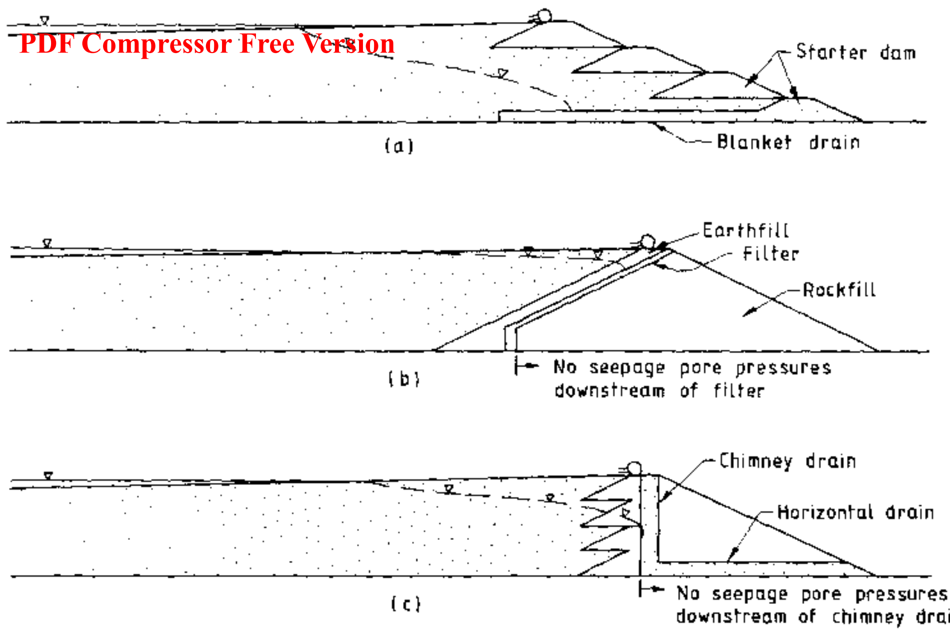


Figure 17.33. Use of internal drainage zones in embankments a) upstream embankment using starter dam wi upstream blanket drain; b) downstream embankment; c) centreline embankment with vertical chimney drain

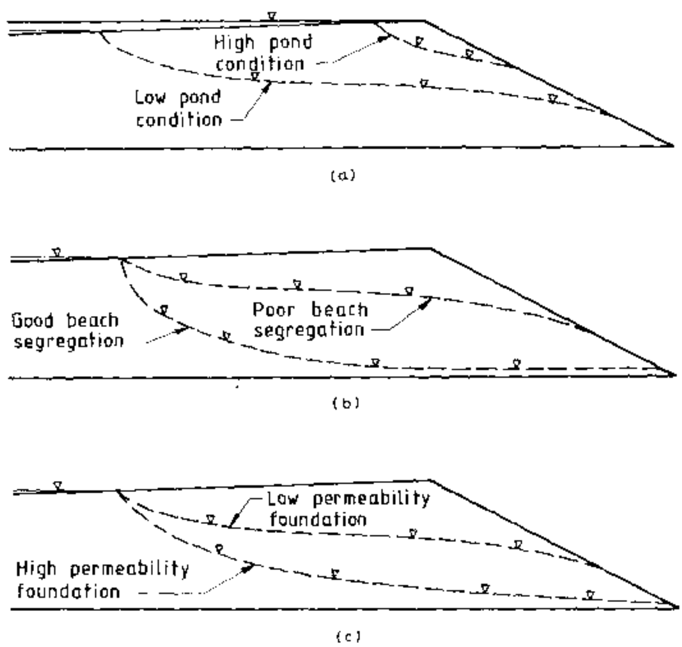


Figure 17.34. Factors influencing phreatic surface location for upstream embankments a) effect of pond wat location; b) effect of beach grain-size segregation and lateral permeability variation; c) effect of foundatio permeability.

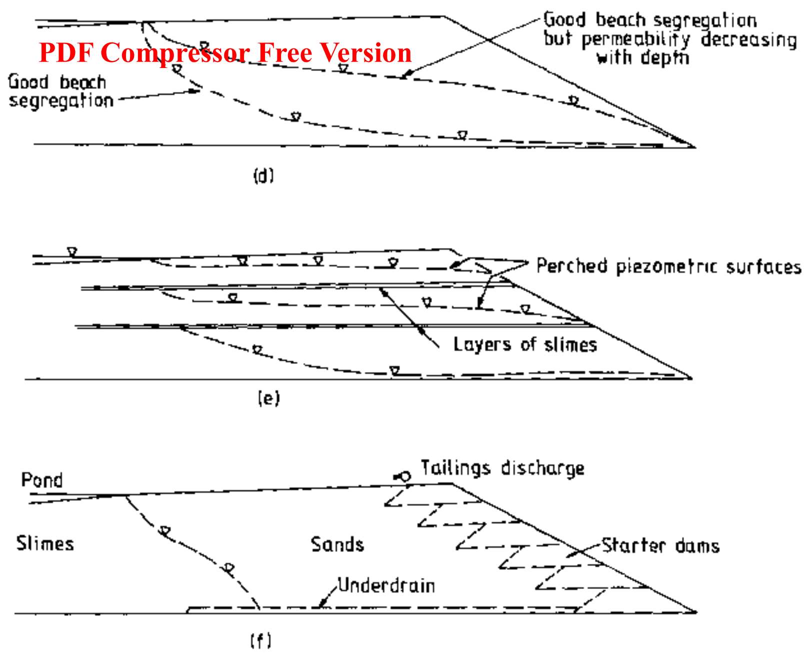


Figure 17.34 (continued). d) effect of decreasing tailings permeability with depth; e) effect of slimes layers; f) effect of underdrains.

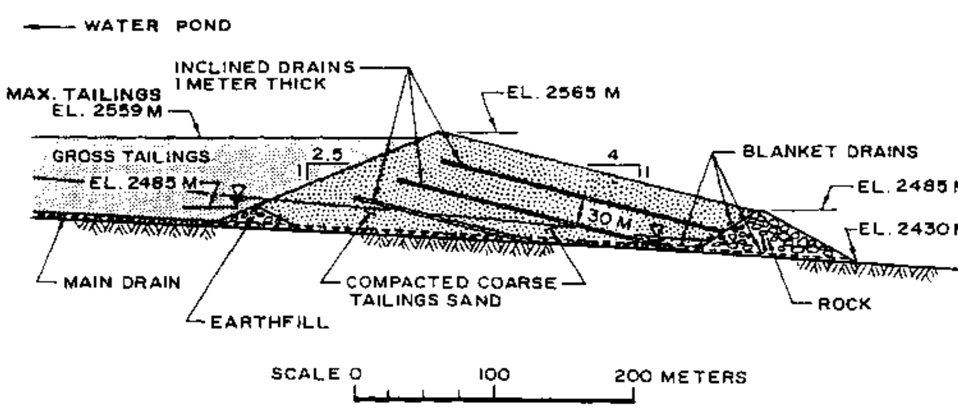


Figure 17.35. Embankment drainage in Perez Caldera No.2 tailings dam (Griffin 1990).

control, followed by the centreline method, and upstream construction. Figure 17.33 shows that by providing internal drainage layers, one can engineer control of seepage. However, this is only achieved at a cost penalty.

Vick (1983) and Blight (1987, 1988) discuss the benefits of sorting of the tailings particles in the beach zone, and how this can lead to a gradation of permeability from high near the discharge point decreasing towards the pond, and, hence, a lower phreatic surface. Figure 17.34

shows this effect as well as the effect of having the water pond near the embankment crest, and the underdrainage effect of a high permeability soil or rock foundation.

Stauffer & Obermeyer (1988) discuss a case where piezometric pressures were lower than might otherwise have been expected, because of the underdrainage effect of a relatively high permeability foundation.

In many designs, drainage zones will be incorporated into the design, either within the

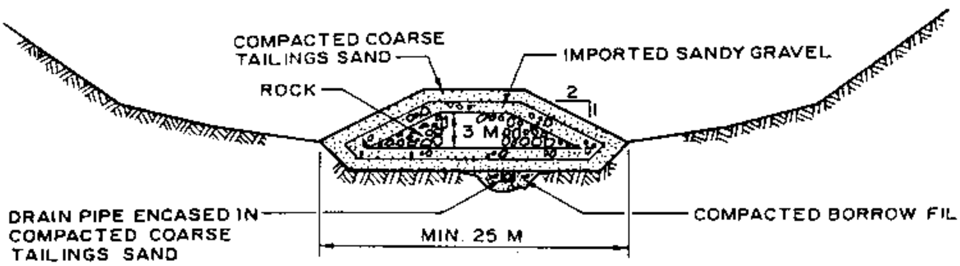


Figure 17.36. Perez Caldera No.2 tailings dam underdrains (Griffin 1990).

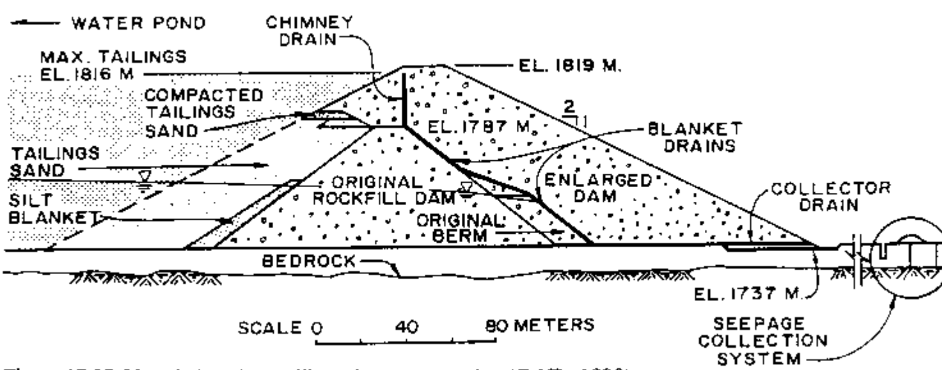


Figure 17.37. Vernal phosphate tailings dam cross section (Griffin 1990).

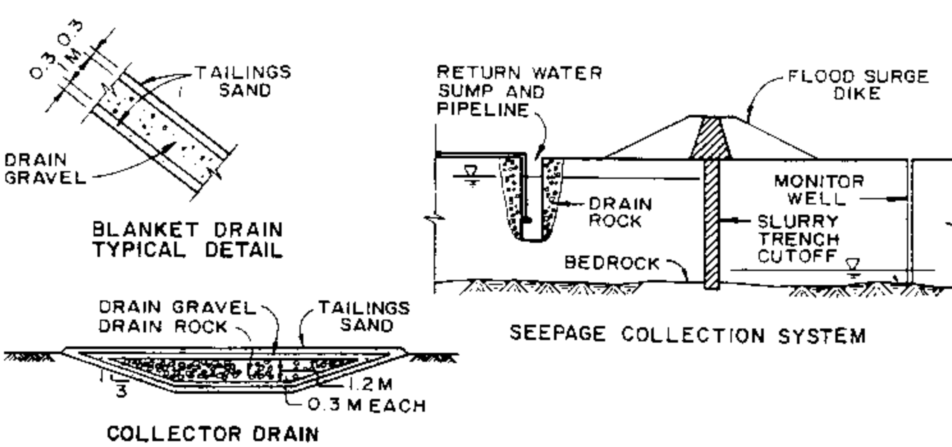


Figure 17.38. Vernal phosphate tailings dam details (Griffin 1990).

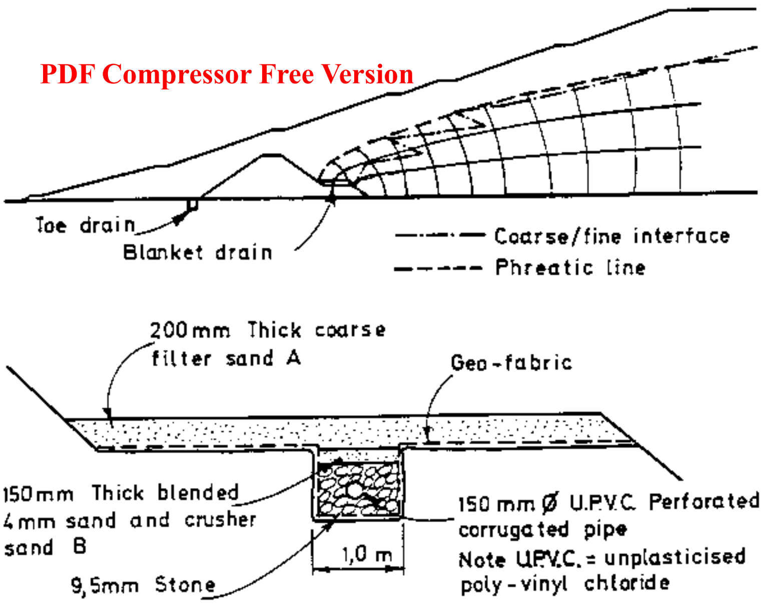


Figure 17.39. Blanket drain (for dam in Fig. 17.27, Lyell & Prakke 1988).

embankment, as shown in Figures 17.35, 17.36, 17.37 and 17.38, or as underdrains under the tailings as shown in Figure 17.39.

As well as controlling the pore pressures in the embankment, the drains under the tailings help accellerate consolidation of the tailings, and with the drains downstream of the dam, facilitate collection of seepage.

17.5.5 *Storage layout*

Some common storage layouts are shown in Figures 17.40 and 17.41.

Some features of these arrangements are:

Ring dike,
- suitable for flat terrain,
- no runoff from external catchments,
- can be staged in plan,
- allows cycling of tailings disposal into separate cells.

Cross valley,
- should be sited at head of valley to limit external catchment,
- usually involves less embankment material than ring dike,
- can be staged either in height or by building multiple impoundments.

Side hill,
- suitable only where hill slopes are relatively flat, say ≯ 10%. Otherwise embankment fill volumes become excessive.

In many cases it is desirable to limit the flow into the storage from external catchments.

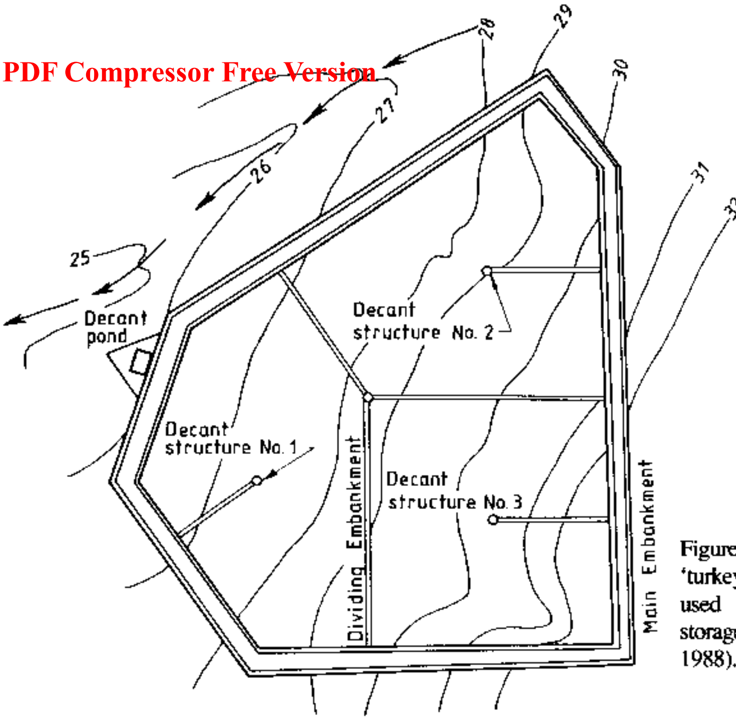


Figure 17.40. Ring dike or 'turkey's nest' configuration as used in Paddy's Flat tailings storage (adapted from Cooper 1988).

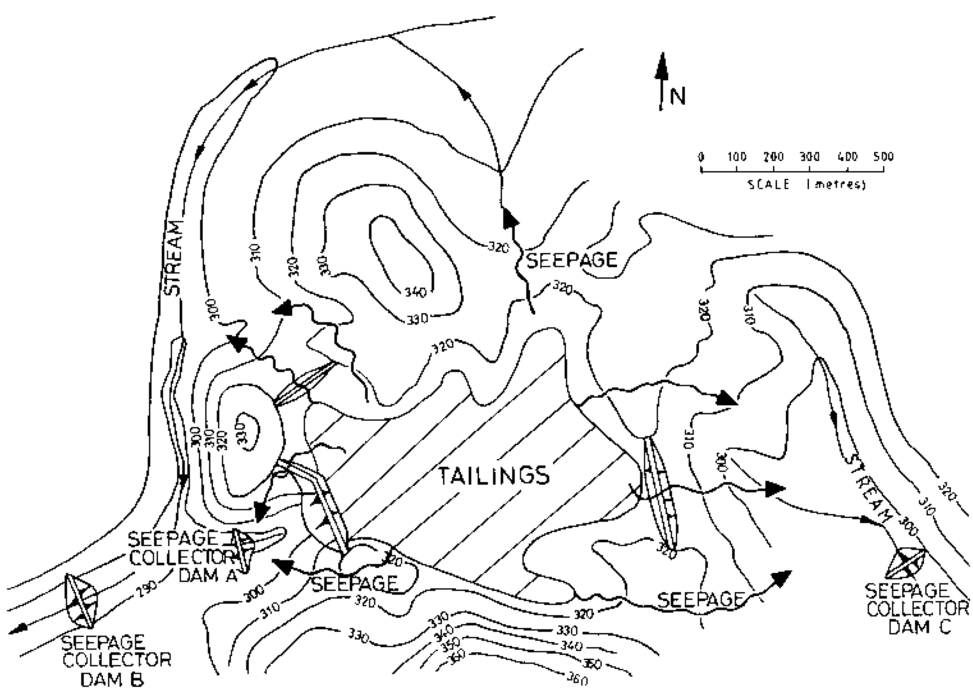


Figure 17.41. Cross-valley impoundment at the head of a small stream.

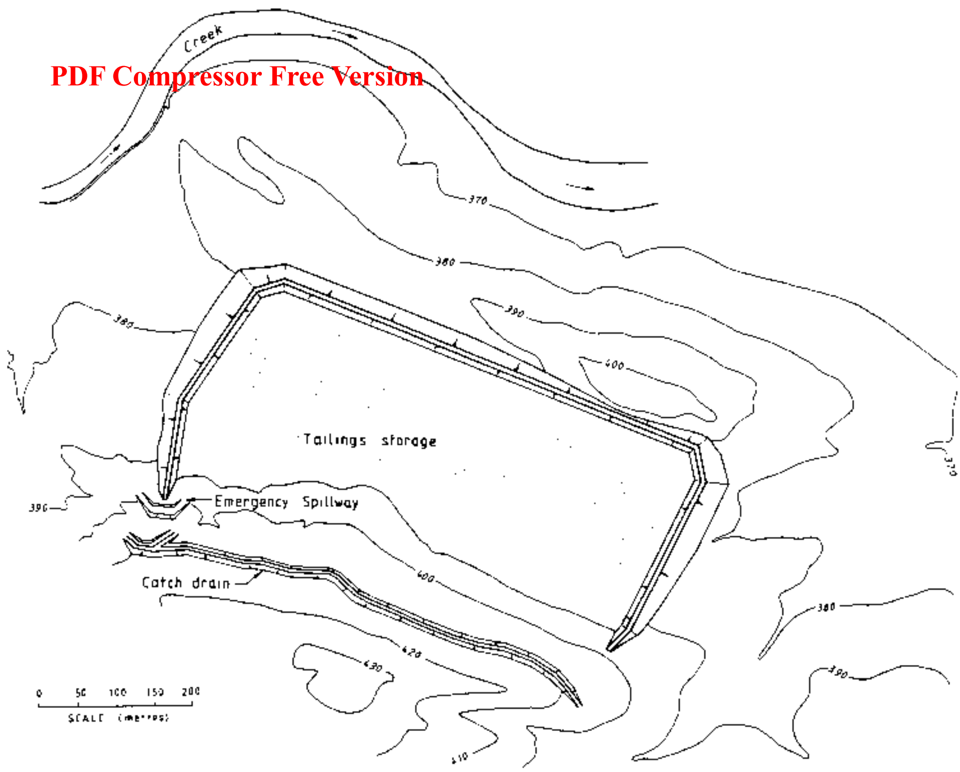


Figure 17.42. Sidehill impoundment as proposed for Ben Lomond tailings dam (Coffey & Partners 1982).

17.5.6 *Other disposal methods*

There are other less commonly used methods of tailings disposal. They are mentioned in the following four sections.

17.5.6.1 *Thickened discharge, or Robinsky method*

In this method tailings are thickened to a higher solids content than would normally be used, i.e. around 60% solids content compared to 30 to 40% for normal tailings operation. At this solids content the tailings can be deposited in a cone shaped deposit as shown in Figure 17.43.

According to Robinsky (1979), the final cone slope is ideally around 6° to limit erosion. The concept is that the greater tailings slope allows storage of larger quantities of tailings. It is claimed that this results in overall reduction in costs, as the saving in embankment costs more than offsets the costs of thickening.

Ritcey (1989) gives details of two mines where the method has been used. Blight & Bentel (1983) discuss the use of thickened tailings to increase the slope of tailings on a conventional dam, to increase the storage capacity. They also give a method for estimating the stable slope of the tailings based on the yield point stress of the slurry.

Vick (1983) and Ritcey (1989) cite studies which have shown that some tailings can liquefy at a slope of 6°, and Vick cautions the use of the method in seismic areas. He points out that the

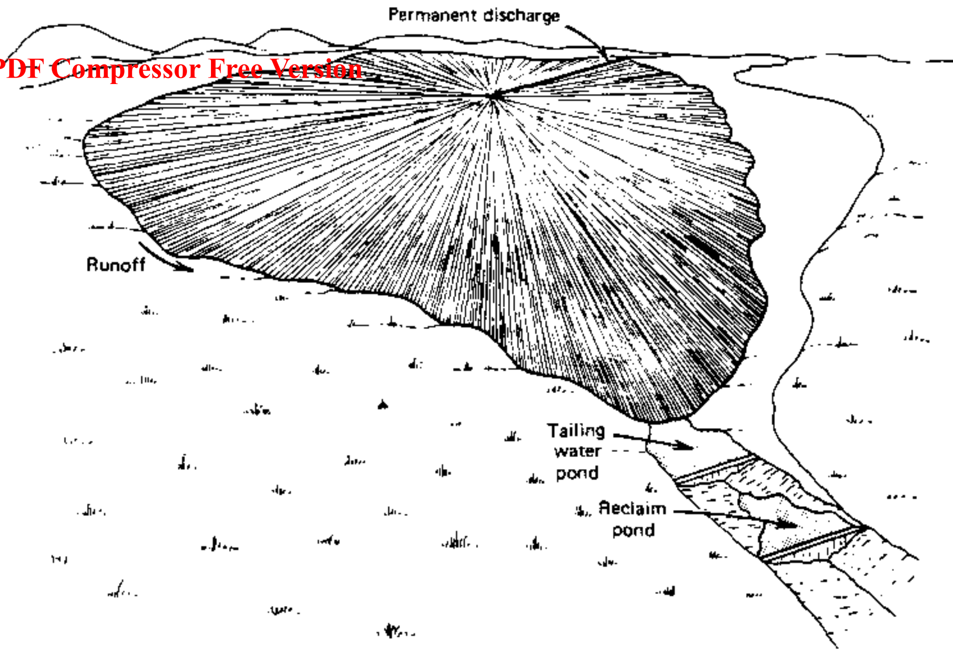


Figure 17.43. Thickened discharge disposal method (diagram from Vick 1983, reprinted from Robinsky 1979).

method can be used only for tailings which would satisfy upstream construction techniques, i.e. tailings with a reasonable sand fraction and a small percentage of clayey fines.

The authors have reservations about its use in most situations, because of the difficulties in achieving the desired degree of thickening and in controlling erosion of the surface.

17.5.6.2 *Disposal into open cut and underground mine workings*

An apparently attractive method of tailings disposal is to discharge the tailings into old open cut or underground mine workings. This presents a low cost method of disposal. However, there are several potential problems. In most open pits the area of the pit is relatively small and results in a subaqueous discharge environment with resultant low densities. This, in turn, leads to high compressibility and low strength. When the area is no longer in use, settlement occurs leaving a water pond. Settlement can total several metres and take tens of years to complete. It may take some years before the tailings dry sufficiently on the surface to allow rehabilitation. Tailings discharged into underground mine workings will only settle to low densities, and very low strength and will be susceptible to flowing out of the workings if, for example, another seam or ore body is removed from below the workings.

Brawner (1979) discusses this, and the use of sandy tailings as 'structural' mine backfill. Thomas (1983) discusses use of tailings for mine backfill.

Vick (1983) discusses the issues involved in underground disposal. Brown et al. (1988) discuss the use of tailings to backfill mine workings.

17.5.6.3 *Discharge into rivers or the sea*

There are some notable examples of this: Bougainville Copper and OK Tedi Mining, Papua

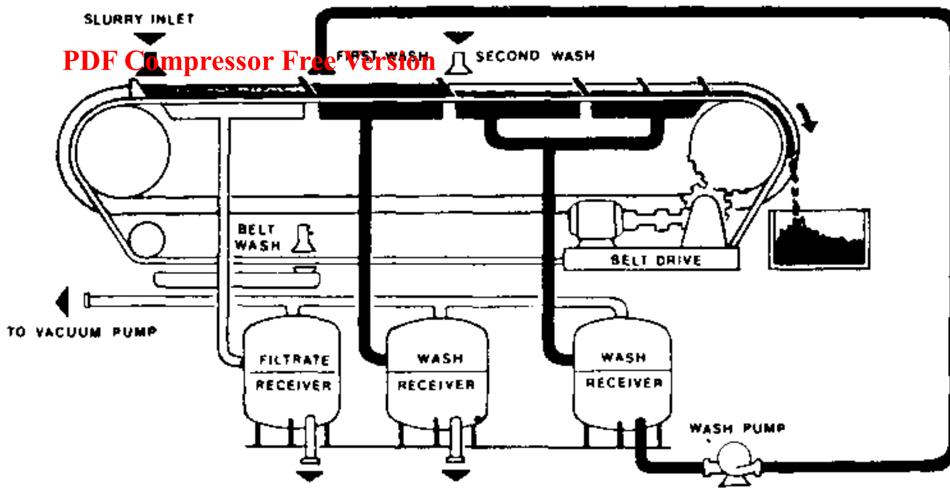


Figure 17.44. Principle of belt filtration (Willis 1984 by permission of Pergamon Press).

New Guinea and the Rosebery Mine, Tasmania. In the latter case, tailings discharge has resulted in devastation of the river due to the chemical content of the tailings water. In Bougainville, considerable sedimentation has occurred resulting in encroachment of the river into adjacent land. As a result of these types of effects, generally speaking, disposal into a river is environmentally unacceptable. The issues are discussed in Vick (1983) and Ritcey (1989).

17.5.6.4 *Belt filtration*

Belt filters are used for dewatering of some tailings to such a consistency that they can be carried by truck for disposal in mine overburden dumps, either separately, or mixed with say coarse rejects. Figure 17.44 shows the principle of belt filtration.

The tailings are spread on a filter cloth supported on a drainage deck, and are dewatered by gravity and the application of a vacuum to the underside of the drainage deck. The process is often aided by large doses of flocculants.

In general, the capital and operating costs of such operations are high, and the success depends on having a uniform quality of tailings. To the authors' knowledge it has only been used where space or environmental constraints preclude other methods.

17.6 SEEPAGE FROM TAILINGS DAMS AND ITS CONTROL

17.6.1 *General*

As discussed in Section 17.2, many mine and industrial tailings have accompanying water or 'liquor' which contains dissolved salts, heavy metals and other residual chemicals from the mineralogical processes. If this liquor escapes to the surrounding surface and groundwater in sufficient quantities, it can lead to unacceptable concentrations, making the water unusable for drinking and affecting aquatic life. Therefore, there is often an emphasis in the engineering of tailings dams on the estimation of seepage rates, and where these prove unacceptable, to the provision of measures to reduce seepage.

Seepage cutoff measures can be very costly, and are often not as effective as the proponents would expect. They can affect the economic viability of a mining project, and certainly the profitability of the operation.

This section presents an overview of the measures which are available and their effectiveness in controlling seepage. The philosophy of accepting that tailings dams will seep regardless of the measures adopted, and collection and/or dilution of the seepage to acceptable concentrations,, is suggested as being more realistic in many cases.

For a more detailed discussion on the topic readers are referred to the chapter by Highland in Vick (1983). Ritcey (1989) discusses chemical and geochemical aspects.

17.6.2 *Principles of seepage flow and estimation*

Many tailings storages (or 'dams') will be constructed on relatively flat land with a deep existing groundwater table. This situation is discussed by Vick (1983), and Figures 17.45 and 17.46 are reproduced from his book.

The following should be noted:

– The rate of seepage flow will be dependent on the permeability of the tailings, the underlying soil and rock, climate, pond operation etc.

– Contaminants in the seepage water will not all join the groundwater. Much will be adsorbed in the foundation soil and rock. Hence, contaminant load does not equal (seepage flow rate) x (contaminant concentration) in the storage.

– Further reduction of contaminant concentration may occur in mixing with stream flows.

– It is contaminant concentration in ground and surface water which is generally critical, not the total quantity. Hence adsorption, dispersion and dilution can yield acceptable water quality in streams or well points, even though the original contaminant levels in the storage may have been unacceptable.

It is important to realise that, in many cases, a partially saturated flow condition will exist in the foundation, at least at the start of operations, and possibly on a permanent basis if the tailings

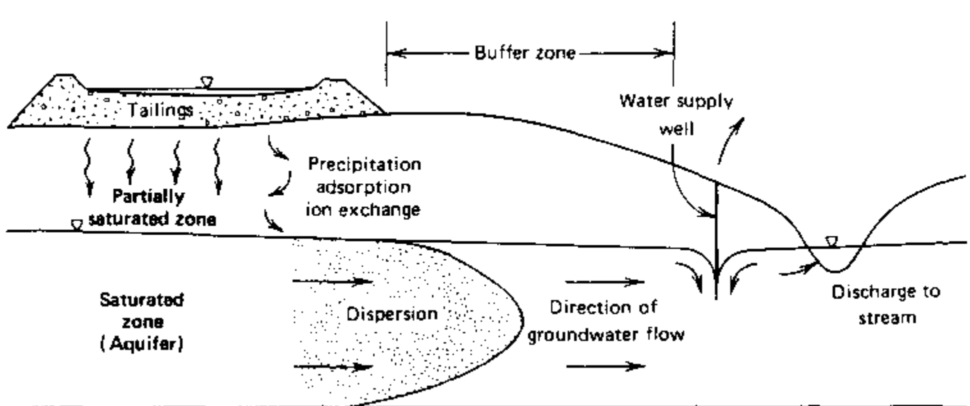


Figure 17.45. Groundwater flow and contaminant transport processes (Vick 1983).

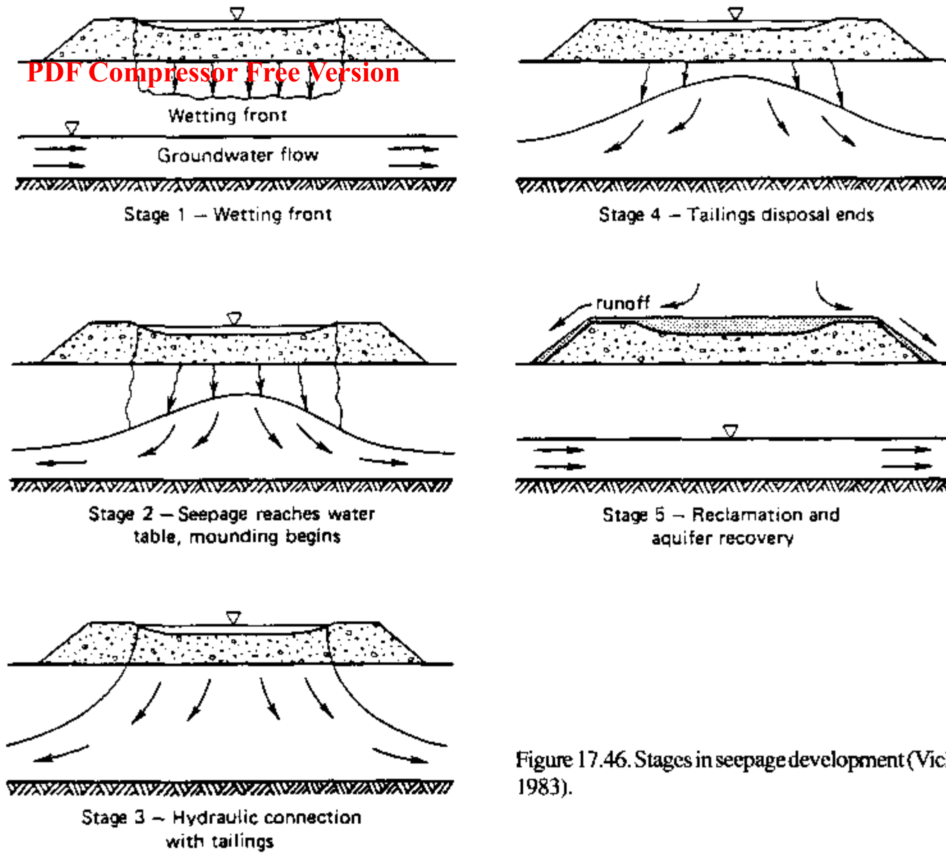


Figure 17.46. Stages in seepage development (Vick 1983).

permeability is low compared to the foundation permeability. Figure 17.46 shows stages in the development of seepage.

Note that flow in the tailings in stages 1 and 2 will be vertical, will not emerge at the toe of the embankment, and will be virtually unaffected by any foundation treatment such as grouting and not intercepted by drains at the toe of the dam.

It may take years for the seepage mound to rise to connect to the tailings (stage 3) or it may never happen.

Figure 17.41 shows an example of a tailings storage which has been constructed at the head of a valley. In this case the final development will consist of several embankments. When estimating seepage from such a storage, it is important to remember that seepage will occur under each of the embankments, and depending on the base groundwater levels, into the hillsides adjacent the embankments. Hence, in Figure 17.41, seepage will occur to the west, north and east but not to the south where natural groundwater levels are higher than the storage. Note that the groundwater does not always mirror the topography and may be affected by local variations in geology, e.g. permeable dykes.

It is the authors' experience that inexperienced engineers and geologists will either forget completely that seepage will occur in all directions from the storage, or at least apply an excessive amount of the site investigation effort and analysis, to the seepage which will flow through and beneath the main embankment.

From the examples shown in Figures 17.45 and 17.46 and 17.41, it will be apparent that the assessment of seepage flow rates will involve:

– Knowledge of the permeability of the tailings, as these are commonly part of the seepage path. In many cases they may control the seepage rates.

– Knowledge of the permeability of the soil and rock underlying the storage, and surrounding the storage. In Figure 17.41 it would be necessary to be able to model the whole of the area between the streams, necessitating knowledge of rock permeabilities well beyond the storage area.

– Modelling of the seepage, usually by finite element methods, which may involve several section models and/or a plan model. This modelling should account for the development of flow as shown in Figure 17.46, and not just model an assumed steady state coupled flow situation, ie. the storage and groundwater coupled as in stage 3, Figure 17.46. A common error is to assume steady state coupled flow, when in fact it does not apply, and to lower the accuracy of seepage modelling further by assuming seepage emerges at the toe of the dam and at ground surface when, in fact, flows downstream of the embankment are inadequate to raise the phreatic surface to ground level.

17.6.3 *Seepage control measures*

Measures to control seepage from tailings storages include:

– controlled placement of the tailings,
– foundation grouting,
– foundation cutoffs,
– clay liners,
– underdrains and toe drains.

17.6.3.1 *Controlled placement of tailings*

In many cases, the most cost effective way of controlling seepage will be to place the tailings so they blanket the base of the storage. Figure 17.47 shows this effect with the tailings forming an effective blanket in b), but not in a) where water is in direct contact with the foundation.

In theory, provided the tailings are of low permeability, they will form as effective a liner to the storage as can be achieved by a compacted clay liner. Vick (1983), for example, shows that tailings with a permeability of 10^{-8} m/sec are as effective as a 2 foot (0.6 m) thick clay liner with a permeability of 10^{-9} m/sec.

It should be remembered that rock in the upper 10 to 30 m of most foundations has a permeability between 1 and 20 Lugeons, or 10^{-7} to 2×10^{-6} m/sec. Most naturally occurring soils will have a similar permeability. Since tailings slimes (even from non oxidised ore) are likely to have a vertical permeability of less than 10^{-7} m/sec, the tailings will often be less permeable than the underlying soil and rock. If the tailings are from oxidised ore or from washeries (such as coal, bauxite, iron ore) they are likely to have a permeability of the order of 10^{-7} to 10^{-9} m/sec or less. Clearly, in these cases, covering the storage with the tailings will be an inexpensive and effective way of limiting seepage.

The effectiveness of the tailings as a 'liner' is dependent on placement methods. If tailings are placed subaerially and allowed to desiccate, lower permeabilities will result from the

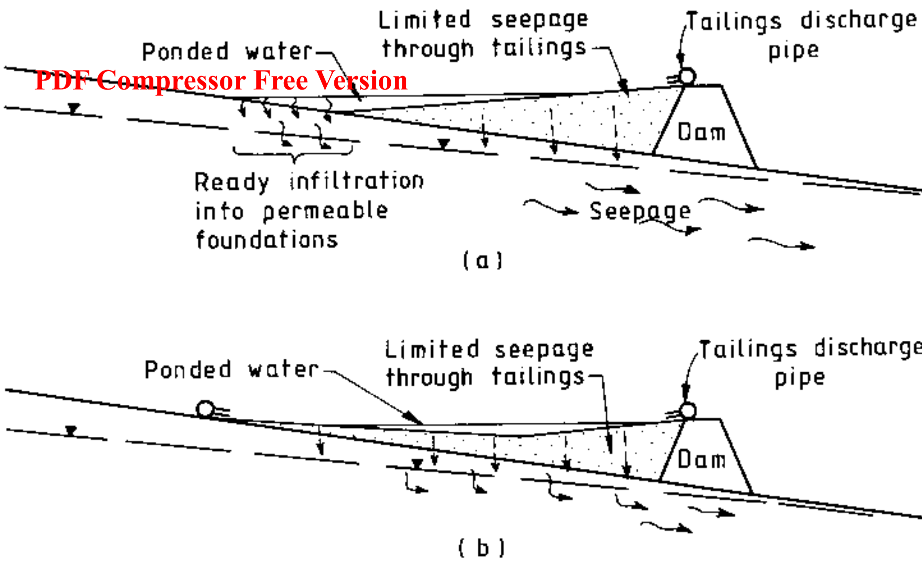


Figure 17.47. Controlled placement of tailings a) tailings not covering foundation; b) tailings covering foundation.

drying, provided cracking does not occur. If placed subaqueously lower densities and higher permeabilities are likely to result.

A potential difficulty with using tailings as a liner, is that the coarser fraction of the tailings tend to settle out more quickly than the fine (or slimes) fraction. Hence a 'beach' of sandy tailings often occurs near the discharge point, and if water is allowed to cover this area subsequently, it can allow local high seepage rates. This can be overcome by shifting the tailings discharge points from one end of the storage to the other, placing slimes under the beach area, and/or by using a liner or seepage collector system under the sandy area.

Seepage can also occur along the contact between tailings and embankment if rock rip rap is used. Another problem is that it can be very difficult to avoid having water pond against the storage foundation, particularly early in the storage operating life.

17.6.3.2 *Foundation grouting*

As discussed in Chapter 12, grouting is not particularly effective in reducing seepage, except in high permeability rock.

In a project on which the authors were involved, the grouting of a 5 km long dam foundation to a depth of about 25 m, on average would have reduced the estimated seepage by only 1%, nearly all of this in a relatively small portion of the foundation affected by faulting, and having a permeability of the order of 100 Lugeons.

It will be seen from the above discussion, that it is unlikely that grouting of tailings dam foundations can be justified on the grounds of reducing seepage. It may be justified on other grounds, such as reducing potential erosion in weathered rock, or where the high permeability zones can be identified from geological information, and only these zones are grouted.

17.6.3.3 *Foundation cutoffs*

For tailings storages constructed on soil foundations, particularly sand or sand and gravel, a significant reduction in seepage may be achieved by construction of an earthfill cutoff or a slurry trench cutoff wall as discussed in Chapter 9.

These cutoffs are of high cost and applicable only in critical situations and where ground conditions allow, i.e. generally soil. They may be applied to extremely weathered rock, e.g. lateritised highly permeable weathered rock.

17.6.3.4 *Clay liners*

Clay liners can be an effective way of reducing the seepage from a tailings storage. For example, if a tailings storage is located in a highly permeable area over sand, and the tailings are moderately high permeability, a clay liner may well be appropriate.

There are some practical aspects which should be considered in the provision of clay liners:

– The permeability of the clay depends on the soil available. For many naturally occurring soils the compacted permeability will be of the order of 10^{-8} m/sec to 10^{-9} m/sec. In the authors' experience, few soils have permeability as low as 10^{-10} m/sec.

– The permeability is affected by the compaction water content and density. To achieve a low permeability, the soil should be compacted to a density ratio of 98% of standard maximum dry density, at a water content of between -1 to +2% of optimum. The permeability can be increased by an order of magnitude by compacting dry of optimum, see Lambe & Whitman (1981).

– The thickness of the clay liner may not be particularly critical, depending on the particular circumstances. It is better to have a relatively thin (say 0.6 m) high quality layer (i.e. good selection, good compaction control) than a thicker, less controlled layer.

– The clay liner is susceptible to cracking on exposure to the sun which can increase its permeability by orders of magnitude. This is particularly critical on sloping sites, where the liner may not be covered by tailings for months after construction. On flat sites, covering the liner with, say, 150 mm of clean sand or silty sand can act to prevent drying of the clay. On sloping sites the sand cover may be eroded by rainfall, and may need to be held in place with a geotextile.

– The permeability of clay liners was questioned in the early 1980's, when some researchers found that the permeability was increased by some orders of magnitude when particular organic leachates were passed through the clay. Later research has shown this is not a problem for water containing inorganic chemicals, even if at high concentrations. Since most tailings liquor would not contain organic leachate, the permeability measured in the laboratory should be a reasonable guide to its long term behaviour.

– If the foundation has openwork gravel or wide open joints, clay liners can be subject to 'sinkhole' development, i.e. erosion of the liner into the underlying foundation.

– The clay liner is quite expensive. A 0.6 m thick liner could be expected to cost of the order of \$ 5/m^2, assuming there was a ready source of clay fill available. Protection against drying and erosion plus surface preparation to give a smooth contour, could be expected to add to this cost giving an overall cost of say 8 to \$ 10/m^2. Because large areas are often involved, the costs can be very large, e.g. a 1 × 1 km area would cost ≈ \$ 10 million to line with clay.

– The clay liner should cover the whole of the tailings storage. Use of a liner as an 'upstream blanket' is unlikely to result in significant reduction in seepage quantities.

– There will be significant seepage through a clay liner. For example, a 0.6 m thick clay liner with permeability of 10^{-9} m/sec would discharge 90 m^3/day over an area of 1 km square under unit gradient.

– The naturally occurring clays in a storage area are unlikely to have a low permeability unless they are excavated and recompacted. *In situ* they are likely to have a permeability of the order of 10^{-5} to 10^{-6} m/sec due to the presence of root holes, fissures etc.

17.6.3.5 *Underdrains*

Drains may be provided under the tailings as shown in Figure 17.48, with or without a clay liner. The drains act to attract the seepage water and discharge it to a collector system, often for recycling to the process plant.

As shown in Figure 17.48 the underdrains may reduce the head on the liner and reduce the seepage. However, the amount of seepage collected and bypassing the collector drains, is dependent on many factors, as follows:

– The spacing of the drains.

– The vertical and horizontal permeability of the tailings.

– The permeability of the liner. Note that there is a nett head on the liner between the drains, and even within the drain, water flowing on the liner will percolate through the liner. If the groundwater level is below the liner, the gradient through the liner will still be at least 1, giving significant seepage.

– The efficiency of removal of water from the drain system.

If no liner has been provided, and the underdrains are laid on the natural ground, there will still be significant seepage into the ground because while the drain has a permeability of say 10^{-5} m/sec the ground will have a significant permeability, e.g. 10^{-6} to 10^{-7} m/sec (if normal weathered rock).

Apart from these design problems, there are some practical aspects:

– The underdrains must be designed to act as a filter to the tailings, or they will rapidly become blocked. Tailings are particularly erodible and will readily clog a poorly designed filter. It is suggested that the criteria proposed by Sherard et al. (1984a, b) be adopted.

– Geotextiles can be used to construct the drains. However, as outlined in Scheurenberg (1982) and Bentel et al. (1982), the geotextile should not be exposed directly to the tailings or it will clog. They overcame this problem by covering the geotextile with a layer of filter sand.

– The underdrains are susceptible to contamination by dry windblown tailings, before they are covered with tailings.

– The seepage water collected in the drains has to be collected and pumped to storage for use in the process plant. This involves expensive collector and pump systems which have to be maintained

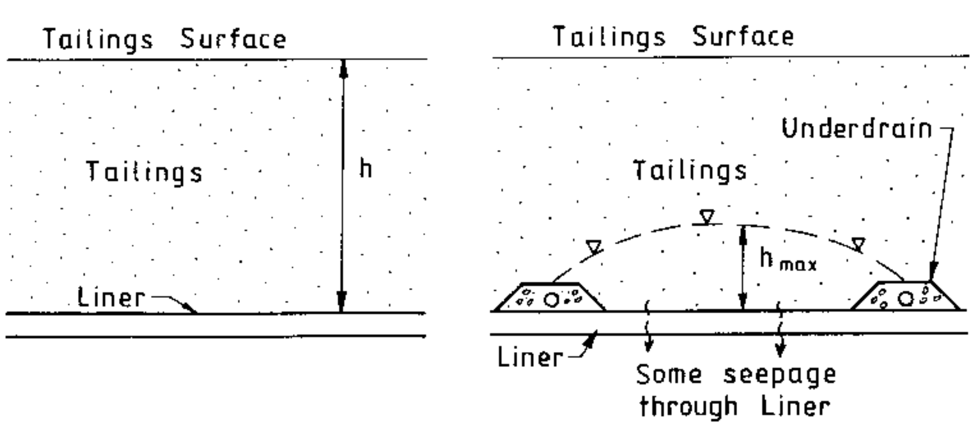


Figure 17.48. Comparison of head acting on clay liner a) without underdrain; b) with underdrain.

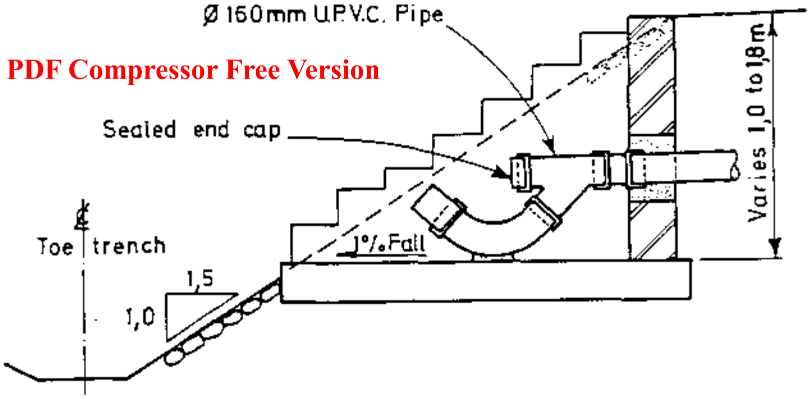


Figure 17.49. Outlet detail for drain to prevent clogging by oxidation (Lyell & Prakke 1988).

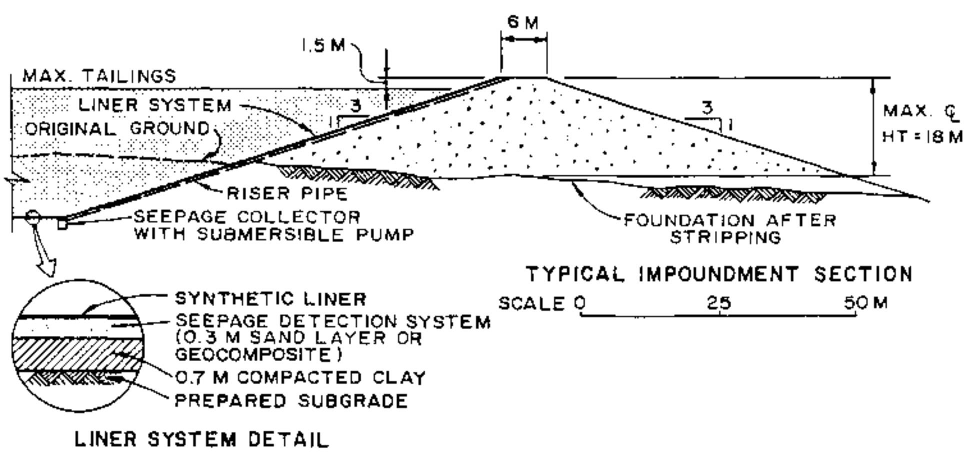


Figure 17.50. Proposed uranium tailings storage (Griffin 1990).

– The filters and outlet pipes can be clogged by oxidation products from the tailings. Figure 17.49 shows an outlet system used by Lyell & Prakke (1988) to overcome this problem.

17.6.3.6 *Synthetic liners*

Synthetic liners are used to line many hazardous waste facilities, often with the provision of drainage layers beneath the first membrane, with a second membrane or clay liner to collect any leachate which leaks past the first. Figure 17.50 shows such a system for a proposed uranium tailings storage in New Mexico, USA.

However, these liners are expensive (a minimum of 15 up to US$ 40/m^2 with drainage systems) and are not appropriate for most tailings disposal situations. There are notable exceptions, e.g. they have been used for lining tailings storages in the Alcoa Alumina Refinery at Kwinana, where the storages are underlain by sands which are used as an aquifer for water supply. In many applications on sloping sites they would be impracticable, because of the need to provide a well graded base, free of irregularities which may penetrate the liner.

It is understood that permeabilities of the order of 10^{-12} to 10^{-14} m/sec are applicable. A 2 mm thick liner with permeability of 10^{-13} m/sec and 15 m of water head on it would give a leakage rate of 60 m^3/day/km^2, i.e. still significant leakage will occur. A common problem is susceptibility to deterioration in sunlight, requiring the liner to be covered.

17.6.4 *Seepage collection and dilution measures*

From the above discussion, it will be apparent that no matter what measures are adopted there is going to be some seepage from tailings storages. This seepage will probably be greatest during operation of the storage, particularly early in the operation when there is little blanketting effect from tailings. It will be exacerbated if water is allowed to pond over the tailings, and adjacent to the natural ground. Seepage will continue after shutdown, even when the tailings are covered, because infiltration of rainfall will occur. Granted this is the case, it will often be more practicable to limit expenditure on seepage 'control' (or 'prevention') measures described above, and design measures to collect the seepage. This may then be pumped back to the storage or the process plant, or diluted with surface runoff before release at acceptable concentrations. Measures which may be taken include:

17.6.4.1 *Toe drains*

A drain may be provided at the downstream toe of the embankment, to collect seepage which emerges at that location.

These drains can be reasonably successful in intercepting seepage, but only if the seepage naturally emerges in this location. In many cases, the flow rates will be such that the phreatic surface stays below the level of the drain. Even when the seepage is sufficient to raise the phreatic surface to flow to the drain, much may still bypass by flowing beneath the drain. Ideally the drain has to penetrate to a low permeable stratum, but this is often not practicable.

Such drains may also intercept surface runoff from the downstream face of the dam and groundwater from downstream, and if the seepage is to be returned to the dam or process plant, may exacerbate water management problems if a 'no release' system is being operated.

17.6.4.2 *Pump wells*

Seepage can be collected by constructing water wells into pervious strata downstream of the tailings embankment, and pumping from these back into the storage or to the process plant.

Such a well system can be reasonably successful in intercepting seepage but there are some disadvantages:

– The pumps lower the piezometric pressures downstream of the storage, so gradients and seepage rates from the storage may be increased.

– The wells also attract water from downstream and so may also add to water management problems in no release operations.

– The wells have to be pumped continuously to be effective, with all the associated costs. The well screens and pumps are susceptible to corrosion and blockage and require maintenance and periodic replacement.

– It is unlikely that it is practicable to operate the wells after shutdown of the storage, so another method may be needed to handle long term seepage.

17.6.4.3 *Seepage collection and dilution dams*

In many cases, a practical way of collecting seepage from tailings storages will be to construct a seepage collector dam or dams. Figure 17.41 shows such a system.

The seepage collector dams may be located sufficiently close to the storage to collect the bulk of seepage, but not too far away so as to limit the external catchment, e.g. Dam A on Figure 17.41. In this case, one would be anticipating pumping the water back into the dam or process system.

Alternatively, one may deliberately locate the collector dam sufficiently far downstream to ensure that the runoff from the catchment to the dam is sufficient to dilute the seepage to acceptable water quality, e.g. Dam B in Figure 17.41.

Whether such an approach is acceptable will depend on the particular circumstances for the tailings storage. For example, it may be unacceptable to have substandard water quality in the stream between Dam B and the tailings storage. Seasonal effects can also be important, e.g. if there is a prolonged dry season, water may pond in the stream, and concentration of contaminants may occur.

The authors' view is that, in many cases, the catch dam with pump-back or dilution may be far more appropriate than expensive measures to control the seepage. From the authors' experience, too many engineers have an over optimistic view of the efficacy of these seepage control measures, or an unrealistic view of what costs a mining operation can reasonably bear to construct such measures.

17.6.5 *Rehabilitation*

After operation, tailings storages must be designed to contain the tailings indefinitely and minimise long term impact on the environment. Factors are:

– long term stability of the 'dam,'
– long term erosion,
– long term effect on groundwater and surface water,
– return of area to productive use.

17.6.5.1 *Long term stability and settlement*

Provided water is drained off the storage, stability and settlement should not be a problem because seepage piezometric pressures are reduced. It is important to ensure that storm runoff cannot overtop the 'dam,' or erosion from creeks etc cannot erode the toe of the 'dam.'

Ponding of water on the tailings storage is likely to occur if tailings have been deposited subaqueously, with resulting low settled density and high compressibility. Figure 17.51 shows the sort of problem which can arise.

In many cases, the presence of a water pond does not create difficulties, but if the water is contaminated it may be unacceptable. In this event, it will be necessary to construct a spillway as shown or to fill the depression with waste rock, more tailings etc. However, this filling will in itself induce additional settlement. Mounding of the tailings and cover prior to shutdown can help alleviate the problem. If tailings are placed subaerially, and well desiccated, they will have dried to a high strength, low compressibility landfill, and provided the surface is adequately contoured, water ponding will not be a problem.

17.6.5.2 *Erosion control*

This can be a major issue. Wind erosion, as well as water erosion, has to be considered. The embankment side slopes are best covered by rock if this is available, otherwise flat (flatter than say 3H to 1V) slopes with good vegetation is required. This is often impossible to achieve on a year round basis.

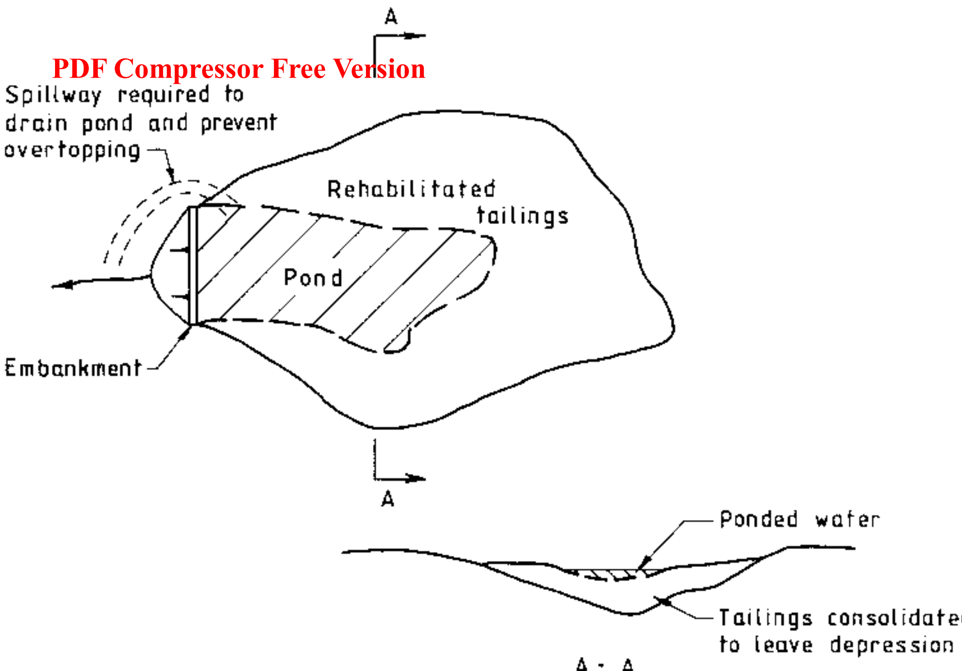


Figure 17.51. Potential ponding after operation ceases.

The tailings surface may be covered with soil, waste rock (or a combination of the two), and vegetated to control erosion and limit infiltration. Some tailings can be successfully revegetated without the need for cover. Cement stabilization has also been used.

It is often necessary to carry out trials during operation to determine what measures will be successful.

Blight (1988) and Blight & Caldwell (1984) describe measures taken to alleviate erosion from abandoned gold tailings dams. Ritcey (1989) also discusses the design of cover. Forrest et al. (1990) and Corless & Glenister (1990) also discuss erosion control measures.

17.6.5.3 *Seepage control*

In many cases the deposited tailings will have contaminants trapped in the accompanying water when operations cease. These contaminants will continue to seep from the tailings as they consolidate, and the water will infiltrate through the cover leaching contaminants as it passes through the tailings.

As was the case during operation, this may or may not be a problem, depending on the concentration of contaminants that reaches groundwater wells or streams in the vicinity. If the contaminants are likely to be a problem, measures will have to be taken to limit infiltration. This can be achieved by redirecting external catchment flows (see Fig. 17.42), contouring the surface of the tailings to encourage runoff, avoiding ponding of water as shown in Figure 17.51, and encouraging transpiration by planting vegetation on the tailings.

It is unrealistic to consider that a 'clay cover' can be provided which will 'seal' the tailings. In the first place, the clay will have a finite permeability even if well compacted and in any case, the clay may be difficult to compact, because the tailings do not form a strong base. The permeability of the cover will be increased by cracking due to desiccation and settlement and penetration by roots and animals.

It is more realistic to design the total system on the assumption that there will be long term seepage, and if the resultant contaminant concentrations are too high, to design seepage collector dams to allow dilution to acceptable concentrations prior to release.

17.6.5.4 *Return of area to productive use*

In principle, it is desirable to return the area to productive use. Usually this can only be for farming or other low intensity usage, not affected by long term settlement.

CHAPTER 18

Monitoring and surveillance of embankment dams

18.1 WHAT IS MONITORING AND SURVEILLANCE?

ANCOLD (1976) give the following definitions:

Monitoring. The observing of measuring devices that provide data from which can be deduced the performance and behavioural trends of a dam and appertinent structures, and the recording of such data.

Surveillance. The continuing examination of the condition of a dam and its appurtinent structures and the review of operation, maintenance and monitoring procedures and results in order to determine whether a hazardous trend is developing or appears likely to develop.

Monitoring and surveillance should be carried out during the construction, first filling and operation of all large and referable dams.

That it is an important issue in embankment dam engineering, can be gauged from the number of ICOLD publications which have been directed to the subject in recent years.

ICOLD (1983a). *Deterioration of Dams and Reservoirs, examples and their analysis.*

ICOLD (1987a). Bulletin 59. *Dam Safety guidelines.*

ICOLD (1988b). Bulletin 60. *Dam Monitoring, General Considerations.*

ICOLD (1988c). Bulletin 62. *Inspection of Dams Following Earthquake – Guidelines.*

ICOLD (1989). Bulletin 68. *Monitoring of Dams and their Foundations.*

Other particularly useful documents which cover this subject are:

ANCOLD (1976). *Guidelines for Operation, Maintenance and Surveillance of Dams.*

ANCOLD (1983). *Guidelines for Dam Instrumentation and monitoring Systems.*

USBR (1983). *Safety Evaluation of Existing Dams.*

FEMA (1987). *Dam Safety: An owners guidance manual.*

National Research Council (1983). *Safety of Existing Dams, Evaluation and Improvement.*

Related to this subject is the definition of what constitutes a 'large dam' and a 'referable dam,' and what is 'hazard' and 'risk.'

For the purposes of inclusion in the ICOLD World Register of Dams, a 'Large' dams are

– all dams above 15 m in height, measured from the lowest part of the general foundation area to the top of the dam;

– dams between 10 and 15 m are included, provided they comply with at least one of the following conditions: (1) the length of the crest, i.e. the top of the dam, to be not less than 500 m; (2) the capacity of the reservoir formed by the dam to be not less than 1 000 000 m^3; (3) the maximum flood discharge dealt with by the dam to be not less than 2000 m^3/sec; (4) if the dam has specifically difficult foundation problems; (5) if the dam is of unusual design.

'Referable' dam. ANCOLD (1976) define this as an artificial barrier, temporary or permanent, including appurtenant works which does or could impound, divert or control water, other liquids, silt, debris or other liquid bearing material and which is: either is 10 m or more in height and has a storage capacity of more than 20 000 m^3; or has a storage capacity of 50 000 m^3 or more and is higher than 5 m.

'Hazard' relates to the potential damage or loss of life in the event of a dam failure, or misoperation of the dam or its facilities. ANCOLD (1983) gives three grades of hazard which are based on US Corps of Engineers criteria: 'Low hazard.' Rural areas where no residences are threatened and economic loss downstream would be minimal, such as farm buildings, limited damage to agricultural land, minor roads, etc.; 'Significant hazard.' Rural areas where a few residences would be threatened and economic loss would be appreciable, including possible damage to secondary roads, minor railways, or relatively important public utilities. 'High hazard.' Where more than a few residences would be threatened, or where loss of human life is liable to be more than a few persons, or where economic loss would be appreciable, such as possible serious damage to extensive community, industrial, commercial or agricultural facilities, highways, primary roads, main railways, important public utilities.

The hazard rating does not relate in any way to whether failure will occur, it is simply a statement of the kind of result if the dam failed.

'Risk' relates to an evaluation of the probability of failure occurring. This may be related to the type of dam, the degree of conservatism of the design, and external influences such as floods and earthquakes.

It should be remembered that the hazard rating can change during the life of a dam. For example development downstream may raise the hazard rating from low to high. The risk may also change, as deterioration of the dam and its appertinent works occurs.

18.2 WHY UNDERTAKE MONITORING AND SURVEILLANCE?

18.2.1 *The objectives*

The objectives of monitoring are (ANCOLD 1983):

– to provide confirmation of design assumptions, and predictions of performance, during the construction phase and initial filling of the reservoir;

– subsequently in the operation phase of the life of the dam to provide an early warning of the development of unsafe trends in behaviour;

– to provide data on behaviour of dams which may not conform with accepted modern criteria, and warrant continuous and close monitoring as a guide to the urgency for introduction of remedial/stabilizing works or other measures;

– during raising or remedial/stabilizing works, which may, of necessity, be carried out with the storage full, close monitoring of structural/seepage behaviour is warranted to ensure that the additional loading introduced by the new works are applied in a manner which will not adversely affect the safety of the dam.

In addition to this may be added:

– to provide data to allow developments in dam engineering, through better measurement of properties e.g. rockfill modulus in CFRD, and checking of analytical methods, e.g. displacements of CFRD face slab; and new construction materials, e.g. asphaltic concrete core, geotextiles.

When setting up a surveillance framework it is vital to ensure that:

– the monitoring data is received by qualified dam engineers and engineering geologists in an ordered manner, so that unusual behaviour can be identified and appropriate action taken.

– the responsibilities of the owner, operator and government authority are clearly defined, with lines of communication established.

18.2.2 *Is it really necessary?*

That it is necessary to have a monitoring and surveillance system established, is highlighted by the number of dams which suffer deterioration and failure. This is discussed in some detail in ICOLD (1983a) and National Research Council (1983).

ICOLD (1983a) studied the approximately 14 700 dams which qualify for the ICOLD register. An extensive survey indicated that of these dams, 1105 (7.5%) had suffered deterioration of one or more type and 107 (0.7%) had failed, resulting in either complete abandonment of the dam, or where damage has been severe, but able to be repaired and the dam brought back into use.

Figures 18.1 and 18.3 to 18.5 and Table 18.1 summarize some of the results of the ICOLD (1983a) survey. Figure 18.2 summarizes all failures (including overtopping which is excluded from the ICOLD (1983a) survey.

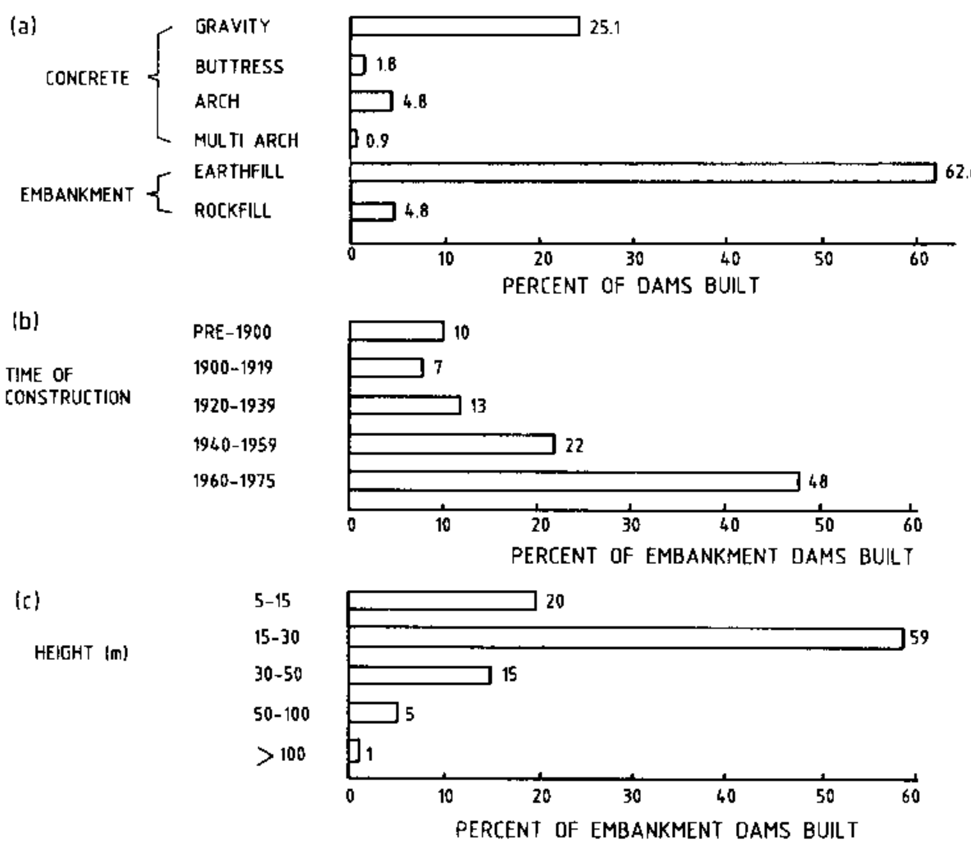


Figure 18.1. Dam types up until 1975 a) by method of construction; b) by time of construction; c) by height (b) and c) for embankment dams only) (adapted from ICOLD 1983a).

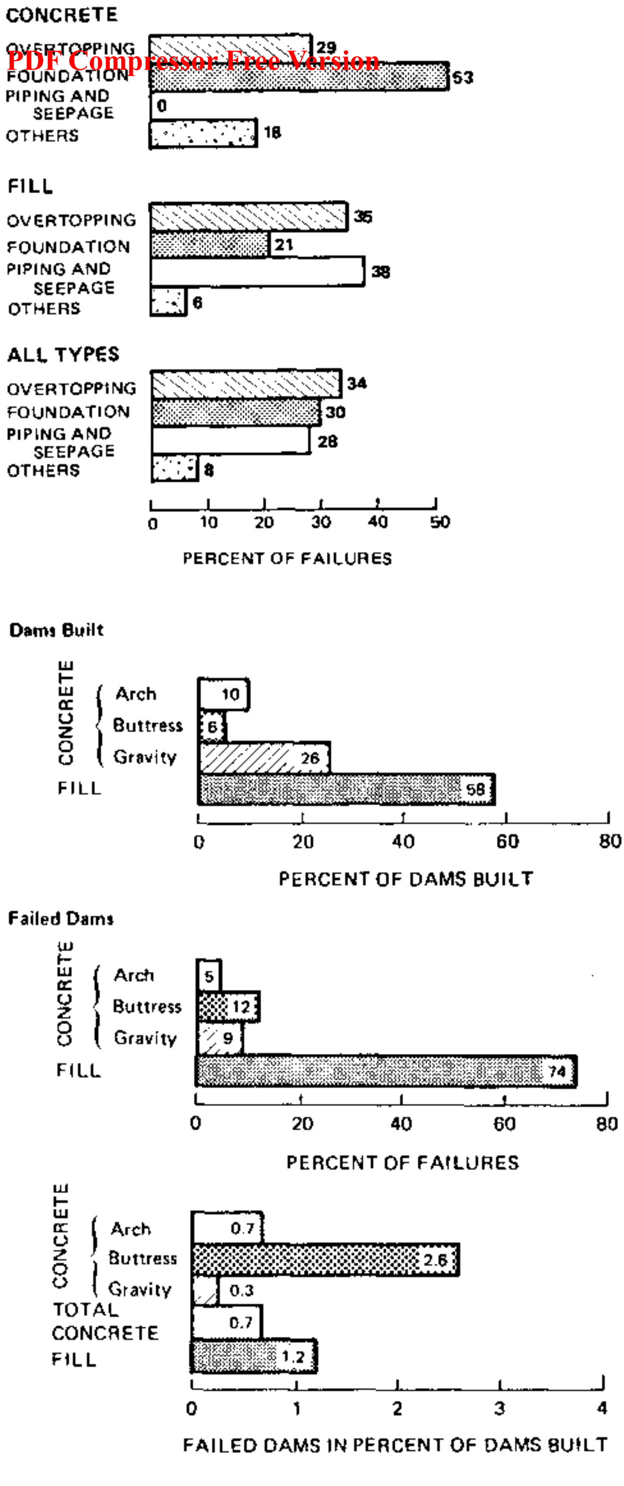


Figure 18.2. Summary of failures of dams and causes of failure of dams over 15 m high (1900-1975) (National Research Council 1983).

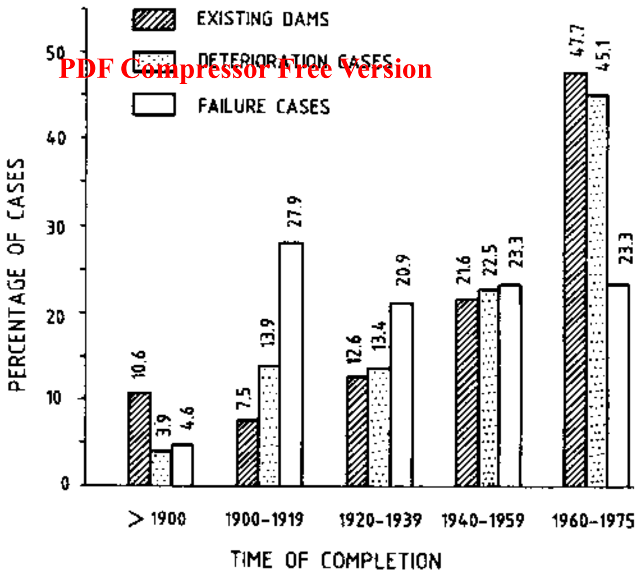


Figure 18.3. Distribution in percentages of existing dams, and deterioration and failure cases affecting the foundations and/or the dam body, by period of completion of the dam (ICOLD 1983a).

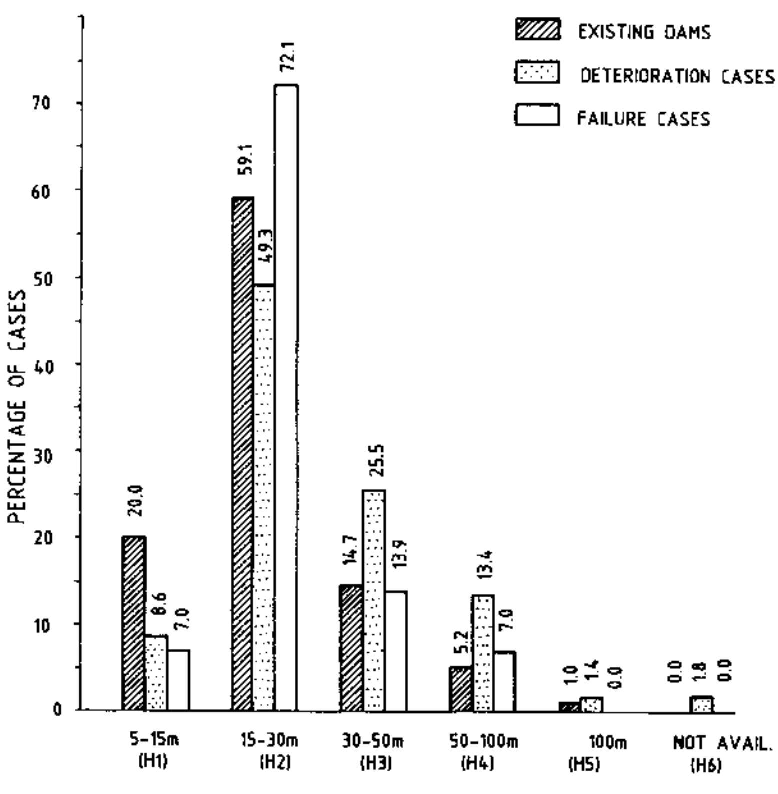


Figure 18.4. Distribution in percentages of existing embankment dams, deterioration and failure cases affecting the foundations and/or the dam body, by height of the dam (ICOLD 1983a).

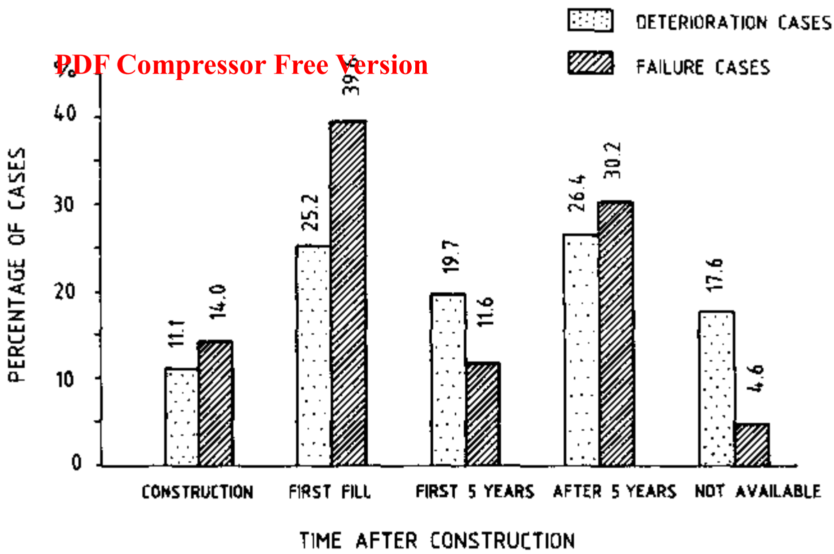


Figure 18.5. Distribution in percentages of deterioration and failure cases affecting the foundations and/or the dam body by deterioration failing time (ICOLD 1983a).

Table 18.1. Most frequent classification of deterioration and dam failure in embankment dams (ICOLD 1983a).

Classification	Percentage of total classification		
	Deterioration	Failure	Failures/deterioration (%)
Percolation (foundation)	27	26	9
Percolation (dam)	25	38	15
Slope protection	19	< 1	NA
Differential movement	17	31	18
Internal erosion (dam)	17	49	29
Downstrream slips	11	16	15
Internal erosioin (foundation)	11	17	15
Compaction	8	12	14
Upstream slips	8	5	6
Bonding between concrete Structures and embankment	7	6	38
Deformation and land subsidence	5	7	14
Watertight systems	4	< 1	NA
Earthquakes	4	< 1	NA
Drainage system and filters	4	< 1	NA
Shear strength	4	5	13
Pore pressure	4	5	11

Notes: 1. NA = not available. 2. 'Percolation' is often related to lack of seepage control measures. 3. Deformation is often associated with percolation, internal erosion, shear strength. Land subsidence is a minor cause.

It can be seen that:

– For all dams, foundation failure, and piping and seepage are the major causes of failure after overtopping.

– Earthfill and rockfill type dams are the most common, and have failure rates slightly higher than that for concrete dams.

– More than 50% of deterioration and failure of embankment dams occur during construction, or on first filling. However, many deterioration failures do occur after the dam has been in service for many years.

– The risk of failure of dams constructed post-1960 is about 5 times less than those constructed pre-1939.

– The percentage of dams suffering deterioration as compared to the number of dams is not improving with time, but the percentage of failures is decreasing, reflecting improved technology of design, construction and monitoring and surveillance.

– More than 79% of all embankment dams are less than 30 m high. These suffer proportionally more failures than higher dams, but less deterioration. This may reflect better monitoring and surveillance of the larger dams.

– It can be seen in Table 18.1 that some types of deterioration are more likely to lead to failure than others, i.e. internal erosion of the dam, and bonding (or lack of it) between concrete structures and the embankment. Others seldom or never lead to failure, e.g. deterioration of slope protection.

18.3 WHAT MONITORING IS REQUIRED?

As pointed out in ICOLD (1989), the type of monitoring needed, the frequency of measurement, and the type of instrumentation depend on the type of dam; at what part of its life it is at, e.g. first filling, or in advanced stages of deterioration; whether the dam was constructed to current design standards; the size of dam and its hazard rating.

ICOLD (1989) differentiates between causal quantities, e.g. water level, and effect quantities, e.g. pore pressures. Table 18.2 gives a checklist under these classifications. Not all are important to embankment dams.

Table 18.2. Monitoring check list

Causal quantities
reservoir water level
rainfall, snowfall
temperature
seismic activity
Effect quantities
seepage
disposal, e.g. crest settlement
deformations, e.g. cracks
pore pressures in dam and foundation
total pressures
seismic accelerations on the dam
vegetation growth/death (due to seepage)
chemical compaction of seepage

Table 18.3a. Index of instrumentation – Monitoring for fill dams (ANCOLD 1983).

ZONED FILL, EARTH AND ROCKFILL, HOMOGENEOUS DAMS, LEVEES ETC.

SIZE OF DAM – HT / Capacity	SMALL 5-10m > 50,000m³			SMALL 10-15m > 20,000m³			MEDIUM 15-100m > 20,000m³			MAJOR > 100m > 20,000m³		
HAZARD RATING	LOW	SIGNIF	HIGH	LOW	SIGNIF	HIGH	LOW	SIGNIF	HIGH	LOW	SIGNIF	HIGH
INSTRUMENTATION / MONITORING SYSTEMS												
1. Visual Inspection	● 3-Monthly	● Monthly	● Monthly	● 3-Monthly	● Monthly	● Weekly	● Weekly	● Daily	● Daily	● Weekly	● Daily	● Daily
2. Seepage Measurement		⊗ T	● T	⊗ T	● S	● S	● T	● S	● S	● T	● S	● S
3. Chemical Analysis of Seepage			⊗		⊗	⊗		⊗	⊗		⊗	⊗
4. Groundwater, Pore Pressure Measurement			⊗		⊗	⊗	⊗	●	●	●	●	●
5. Surface Displacement Measurement					⊗	⊗	⊗	●	●	●	●	●
6. Internal Displacement Measurement								⊗	⊗		●	●
7. Internal Stress Measurement												
8. Hydrometeorological								⊗	⊗		⊗	⊗
9. Seismological											⊗	⊗
10. Telemetering Automatic Recording								⊗	⊗		⊗	⊗

LEGEND
● INDICATES MONITORING BY INSTRUMENTATION/ SYSTEM TO BE CONSIDERED
⊗ REF. NOTE 2

MONTHLY, DAILY- FREQUENCY OF VISUAL INSPECTION
"T"- INDICATES TOTAL DISCHARGE MEASUREMENT
"S"- INDICATES DISCRETE MEASUREMENT

NOTES: –
1 FREQUENCIES OF INSPECTION SHOWN ARE FOR THE IN SERVICE PHASE ASSUMING DAM MOVEMENTS AND BEHAVIOUR HAVE SENSIBLY STABILIZED. DURING FIRST FILLING AND INITIAL SERVICE YEARS MORE FREQUENT INSPECTIONS APPLY
2 IN UNUSUAL CIRCUMSTANCES WHERE THE DAM MAY
- WARRANT CONTINUOUS STRUCTURAL MONITORING
- HAVE INADEQUATE SPILLWAY CAPACITY
- BE IN A HIGH SEISMIC RISK AREA
- HAVE FOUNDATION PROBLEMS (STRATIFIED FOUNDATIONS, STEEP ABUTMENTS ETC.)
- INVOLVE A JUNCTION OF FILL AND CONCRETE WALLS
- BE BUILT OF UNUSUAL MATERIALS

ADDITIONAL MONITORING ⊗ TO BE CONSIDERED

Table 18.3b. Index of instrumentation – Monitoring for deck type dams (ANCOLD 1983).

	CONCRETE OR ASPHALT FACED ROCKFILL - WATER RETAINING MEMBRANE ON U/S FACE OF EMBANKMENT								
SIZE OF DAM	SMALL						MEDIUM			MAJOR		
- HT / Capacity	5–10m > 50,000m³			10–15m > 20,000m³			15–100m > 20,000m³			> 100m > 20,000m³		
HAZARD RATING	LOW	SIGNIF	HIGH	LOW	SIGNIF	HIGH	LOW	SIGNIF	HIGH	LOW	SIGNIF	HIGH
INSTRUMENTATION / MONITORING SYSTEMS												
1. Visual Inspection	● 3-Monthly	● 3-Monthly	● Monthly	● 3-Monthly	● Monthly	● Monthly	● Monthly	● Daily	● Daily	● Weekly	● Daily	● Daily
2. Seepage Measurement		⊗ T	● T	⊗ T	● T	● T	● T	● T	● T	● T	● T	● T
3. Chemical Analysis of Seepage					⊗	⊗		⊗	⊗		⊗	⊗
4. Water Table Measurement												
5. Surface Displacement Measurement					⊗	⊗	⊗	●	●	⊗	●	●
6. Internal Displacement Measurement												
7. Internal Stress Measurement												
8. Hydrometeorological								⊗	⊗		⊗	⊗
9. Seismological											⊗	⊗
10. Telemetering Automatic Recording								⊗	⊗		⊗	⊗

LEGEND ● INDICATES MONITORING BY INSTRUMENTATION/ SYSTEM TO BE CONSIDERED

⊗ REF. NOTE 2

MONTHLY DAILY- FREQUENCY OF VISUAL INSPECTION
T- INDICATES TOTAL DISCHARGE MEASUREMENT
S- INDICATES DISCRETE MEASUREMENT

NOTES:-

1. FREQUENCIES OF INSPECTION SHOWN ARE FOR THE IN SERVICE PHASE ASSUMING DAM MOVEMENTS AND BEHAVIOUR HAVE SENSIBLY STABILIZED. DURING FIRST FILLING AND INITIAL SERVICE YEARS MORE FREQUENT INSPECTIONS APPLY (REF. SECTION 9)
2. IN UNUSUAL CIRCUMSTANCES WHERE THE DAM MAY
   - WARRANT CONTINUOUS STRUCTURAL MONITORING
   - HAVE INADEQUATE SPILLWAY CAPACITY
   - BE IN A HIGH SEISMIC RISK AREA
   - HAVE FOUNDATION PROBLEMS (STRATIFIED FOUNDATIONS, STEEP ABUTMENTS ETC.)
   - INVOLVE A JUNCTION OF FILL AND CONCRETE WALLS
   - BE BUILT OF UNUSUAL MATERIALS

   ADDITIONAL MONITORING ⊗ TO BE CONSIDERED

The several ICOLD publications listed above give very little guidance as to the level of monitoring required for any particular dam. Most of the discussion in ICOLD (1989) relates to the 20% of dams which are higher than 30 m, in fact most relates to the 6% which are higher than 50 m. The Norwegian report in ICOLD (1989) indicates that, because most of their dams have been of similar design and constructed on good foundations, only 23 out of 150 dams have had any instrumentation other than:

– measurement of leakage,

– measurement of surface deformations of the dam,

– and sometimes measurement of construction pore pressures (core material is often wet glacial till).

ANCOLD (1983) gives a useful guides based on the size of dam and hazard rating. These are reproduced in Tables 18.3a and b.

It will be seen that there is a great reliance on visual inspection, and seepage measurement and observation for smaller dams, reflecting the reality that it is not economically feasible to heavily instrument such dams. For medium sized dams, pore pressure and surface displacement are added.

Table 18.4 summarizes the detection methods which led to the identification of deterioration of embankment dams in the ICOLD (1983a) study.

It will be seen that visual inspection and seepage, followed by pore pressure and surface displacement measurements account for most of the deterioration detected. To a certain extent this will be because, often, only those activities were carried out, but it does show that such procedures are likely to be adequate, at least for dams less than 30 m high.

Figures 18.6 to 18.9 show examples of quite intensive monitoring for Prospect Dam (a 100

Table 18.4. Number of applications of detection methods used to detect deterioration in embankment dams (ICOLD 1983a).

Detection methods	Deteriorations affecting foundations	Deteriorations affecting dam body	Deteriorations affecting foundations and dam body	Total No	Total %
Direct observation	92	250	59	401	63
Water flow measurements	37	30	17	84	13
Phreatic level measurements	8	2	2	12	2
Uplift measurements	1	–	–	1	< 1
Pore pressure measurements	17	8	5	30	5
Turbidity measurements	3	1	2	6	1
Chemical analysis of water	3	1	–	4	1
Seepage path investigations	1	5	2	8	1
Horizontal displacement measurements	3	25	2	30	5
Vertical displacement measurements	5	27	7	39	6
Strain measurements	–	1	1	2	< 1
Rainfall measurements	–	1	–	3	< 1
Sounding investigation	2	1	–	3	< 1
Design revision (new criteria)	1	3	–	4	< 1
Not available	3	3	1	6	1
Total of deterioration cases	109	260	63		

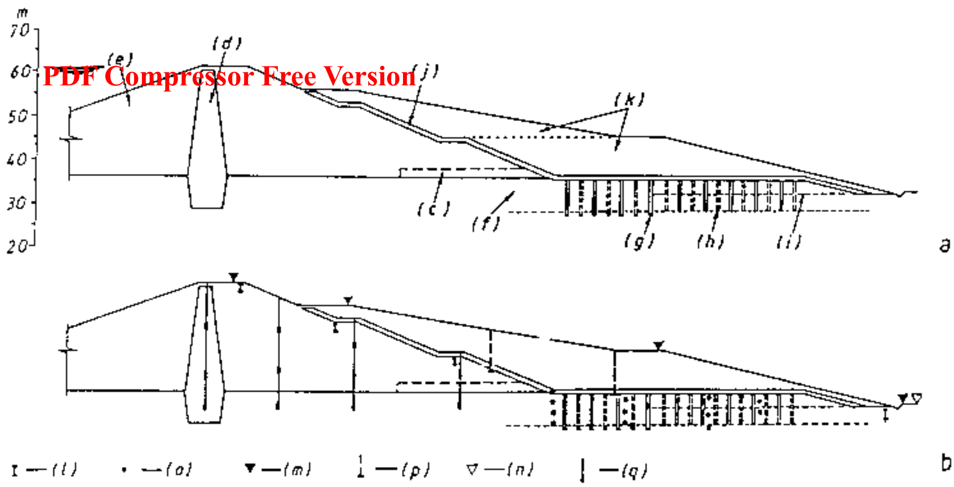


Figure 18.6. Monitoring for Prospect earthfill dam (ICOLD 1989). a) Cross section showing old dam and fill buttressing. b) Cross section showing monitoring systems. c) Rubble filled drainage tunnel. d) Puddle clay core. e) Clay shoulders. f) Alluvial clay foundation. g) Filter drainage cutoff trench along downstream foundation extending to rock. h) Vertical sand drains. i) Lateral drains. j) Filter zone. k) Stabilizing fill material (shale) constructed in two stages. l) Inclinometers. m) Surface settlement points. n) Seepage collection and measurement. o) Pneumatic and electric piezometers. p) Foundation settlement installations. q) Open standpipe piezometers.

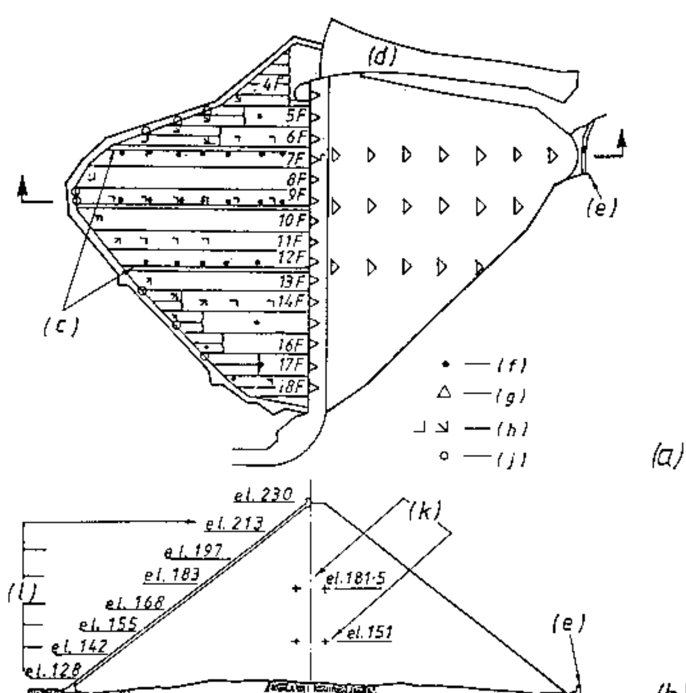


Figure 18.7. Monitoring for Cethana concrete face rockfill dam (ICOLD 1989). a) Plan view. b) Cross section. c) Inclinometer pipes. d) Spillway. e) Leakage weir. f) Anchor points for wires. g) Survey targets. h) Strain gauges. j) Perimetric joint meters. k) Hydrostatic settlement cell. l) Installation level of strain gauges.

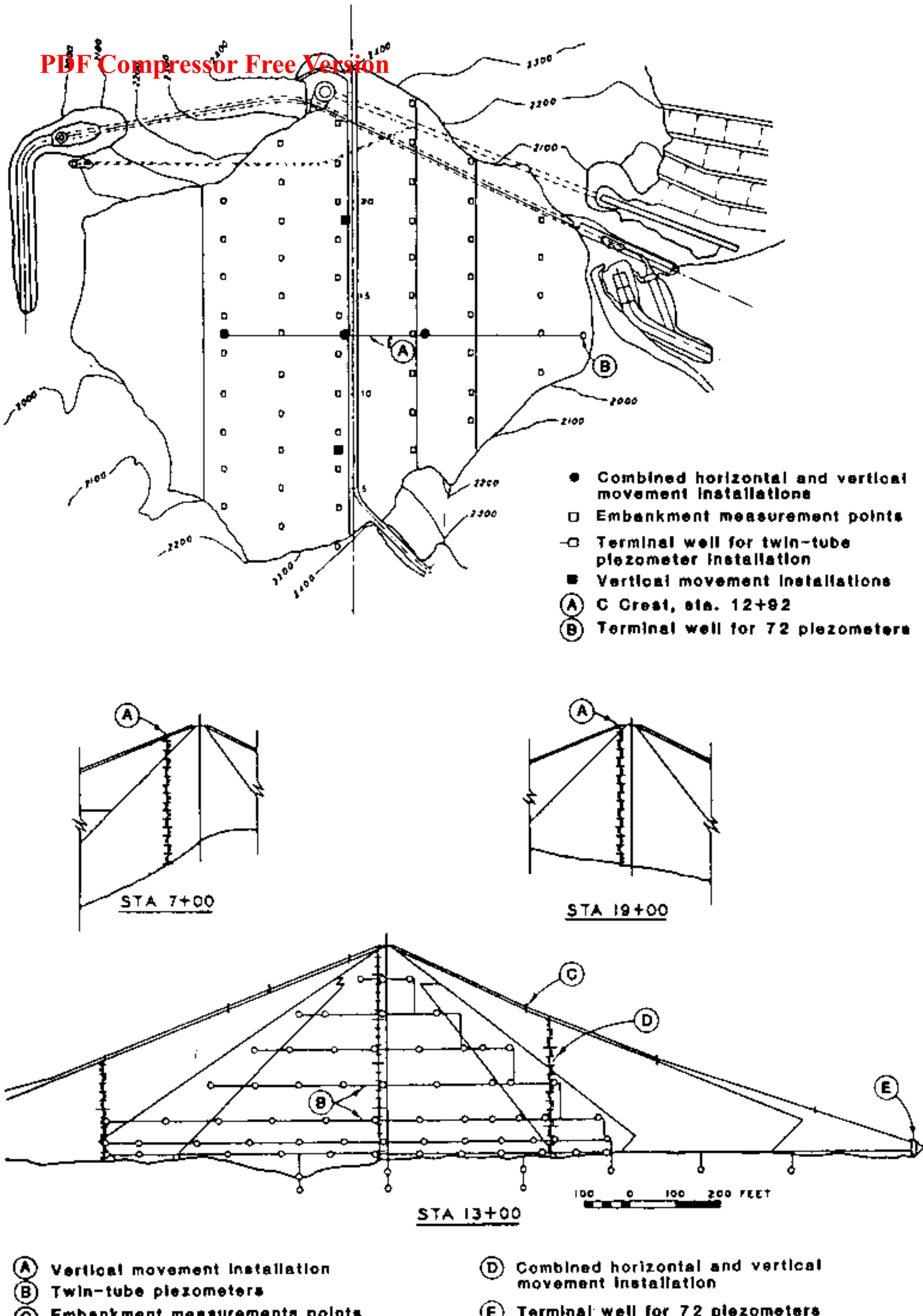


Figure 18.8. Monitoring for Trinity earth and rockfill dam (ICOLD 1989).

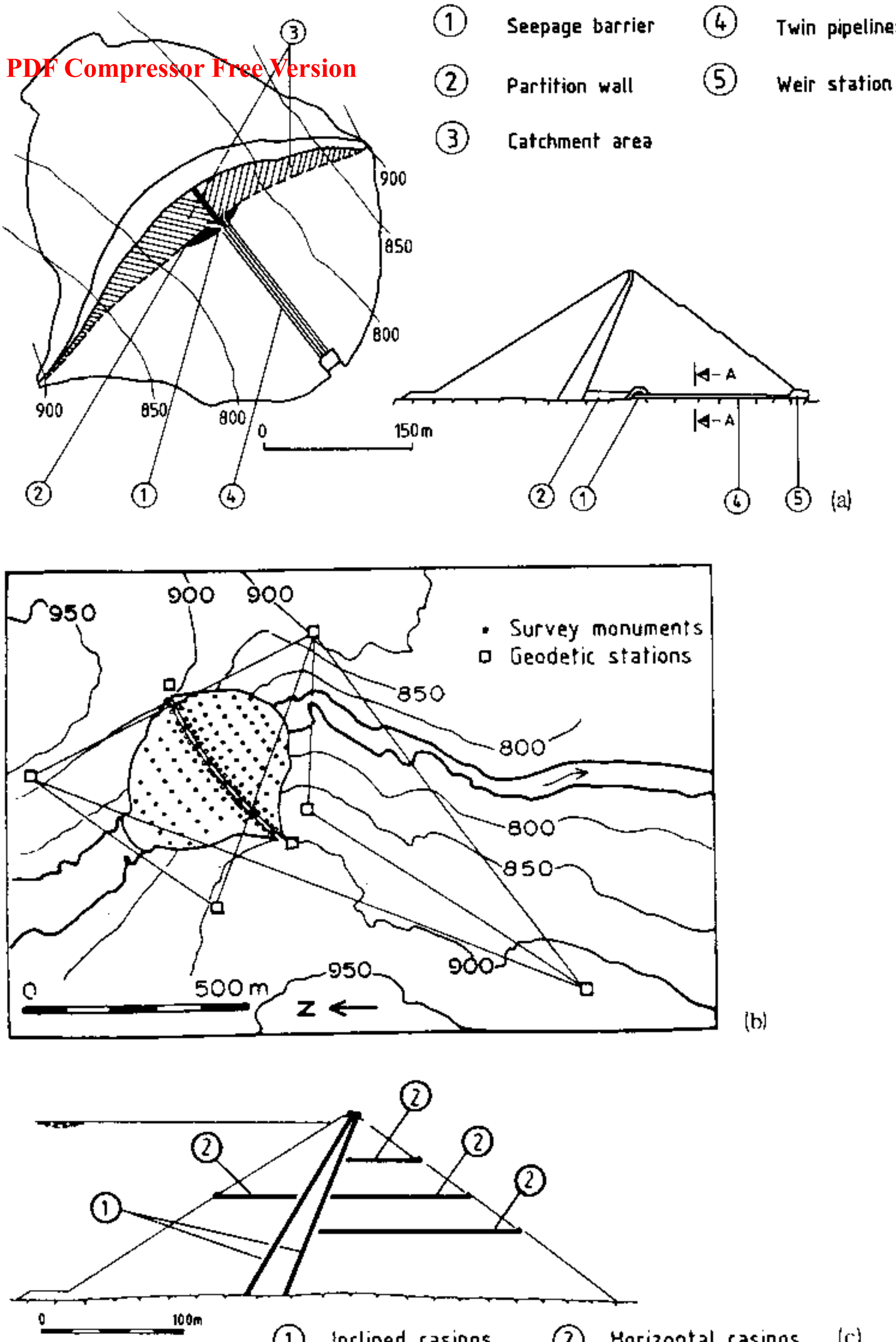


Figure 18.9. Monitoring for Svartevann earth and rockfill dam a) leakage measurement; b) surface displacement; c) internal displacement and strains.

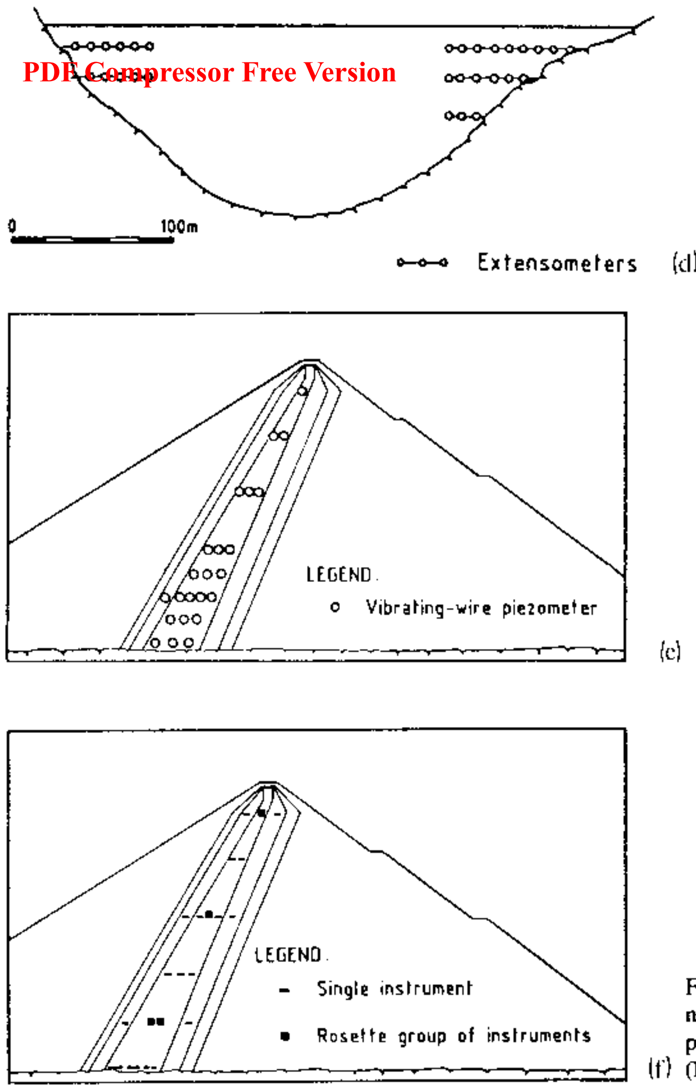


Figure 18.9 (continued). d) internal displacement and strains; e) pore pressures; f) earth pressure (ICOLD 1989).

year old 35 m high puddled core dam, constructed of erodible soil with no filters in the original construction); Cethana Dam, a 110 m high CFRD; Trinity Dam (150 m high earth and rockfill); and Svartevann Dam (129 m high earth and rockfill) (all from ICOLD 1989).

18.4 HOW IS THE MONITORING DONE?

18.4.1 *General principles*

Some general principles on dam monitoring are given by the French National Committee in ICOLD (1989): '(a) Dams are structures with a long life span. *Reliable* instruments with a similar life span are necessary, and they must be accessible, verifiable and replaceable. (b) They must be *sensitive* to give early warning of sudden changes or trends that are very small but may be significant indicators of distress. (c) They should be *simple* so that frequent readings can be made quickly by site or operating staff without the need for specialists; in this way, the dam is more or less permanently monitored, and the records are available for statistical analysis. (d) In recent decades far more attention has been focused on deformations and hydraulic behaviour in the *foundations*. (e) *Visual inspection* and surroundings by people who know the dam is very important because even the best instruments will not find fissures, leakages, damp spots or their growth, etc. (f) The choice of parameters to be measured and the positioning of instruments cannot always be optimal at the design stage. The number of instruments sometimes amazes but is justified by the need for effective monitoring of an as yet unfamilar structure. Optimisation of the system can be made by abandoning some instruments or installing others as the real behaviour becomes better understood. (g) Although modern information processing can help with the processing of huge masses of raw data, experience has shown that it is a good policy to read often a limited number of key instruments very well situated in the dam. (h) The first filling of a reservoir is a stage in its life that is both important and delicate. It is in fact the proof that a dam can fulfil its design functions. Of course, measurements must start earlier so that the structure's initial state is known and some instruments are read during construction so that the reaction of dam and foundation to loading are known. (i) Continuous reading of some instruments is conceivable during first filling to obtain a large amount of information during this fairly short period. During normal operation the raw data is processed by computer to give useful information. However complete automation – the automatic transmission of the state of safety of a dam and possibly of danger signs – is not used by EDF. The delicate task of fine interpretation belongs to the engineer.'

Many of these statements are repeated by the other national committees who reported in ICOLD (1989). Several suggest caution in using new instruments, preferring to introduce them in conjunction with older, proven instruments, rather than alone.

18.4.2 *Visual inspection*

A visual inspection should be carried out by the reservoir operator and at less frequent intervals by dam engineers. The inspection should be carried out to a check list. An example of such a check list is given in Table 18.5. The checklist is for an earth dam, but could be modified for other types of dams. Seepage measurement would normally be included on the same report sheet. USBR (1983) and FEMA (1987) have more detailed check lists.

18.4.3 *Seepage measurement and observation*

Seepage data is one of the best indicators of a dam's performance. By observing the location, quantity and quality of seepage emerging from the dam embankment and its foundation, and particularly the changes which occur, one can get early warning of problems which may be developing, particularly in the important problem areas of internal erosion in the dam and its abutment, and in increased pore pressures.

It is emphasised that routine observation of where seepage is emerging can be as useful a guide as the actual measurement. However, as indicated by most of the reporting countries in ICOLD (1989) and ANCOLD (1983), it is usual to measure the quantity of seepage.

As shown in Figure 18.10, it is preferable to collect the seepage close to the downstream toe

Table 18.5. Checklist of conditions to be noted on visual inspection (adapted from ANCOLD 1976).

(a) Vegetation on dam and within 15 metres beyond toe of dam
- Overgrowth: Requiring cutting for dam surveillance, requiring weed control for dam surveillance, indicating seepage or excessive capillarity
- Wet terrain vegetation: Watch for boils, watch for sand cones, deltas, etc., changes with the season, pond level changes
- Incomplete – Requiring repair: Poor growth, destroyed by erosion

(b) Drainage ditches
- Clogged with vegetation
- Damp
- Flowing water: Quantity
- Boils
- Silt accumulations, deltas, cones

(c) Embankment
- Freeboard-pond level
- Crest: Cracking, subsidence
- Upstream face: Cracking, surface erosion, gullying, wave erosion
- Downstream face: Cracking, subsidence, bulging, erosion, gullies; depth, moisture on dry days; damp areas, boils, seeps
- Berms and within 15 metres beyond toe of dam: Erosion, gullies, damp areas, boils, seeps

(d) Spillways
- Intake level, boards
- Intake structure
- Discharge conduit condition
- Seepage or damp areas around conduit
- Erosion below conduit
- Boils in the vicinity of conduit
- Spillway slabs for uplift, subsidence, cracking

(e) Areas of previous repair
- Effectiveness of repair
- Progression of trouble into new area

of the impervious zone, and to isolate areas from each other so the readings are not influenced excessively by flow through rockfill zones and runoff from abutments.

However, as pointed out by ANCOLD (1983), for smaller dams it is acceptable to measure total seepage downstream of the toe of the dam.

Measurement of seepage may be made by:

– V-notch or similar measuring weirs. This may include continuous recording, and telemetering of data for more important dams.

– Level switches may be provided to give an independent alarm.

– Timed discharge into measuring vessels.

– Visual inspection where flow rates are very small.

It is often impractical to collect and measure all seepage, particularly for dams on alluvial foundations. In these cases, installation and monitoring of piezometers in the foundations under and downstream of the dam can give information on changing conditions, which might indicate a problem is developing. Problems also are experienced where the toe of the dam is flooded by the tailwater from a hydropower station or irrigation outlet. In these cases collecting and measuring seepage in the body of the dam, as shown in Figure 18.10, is desirable.

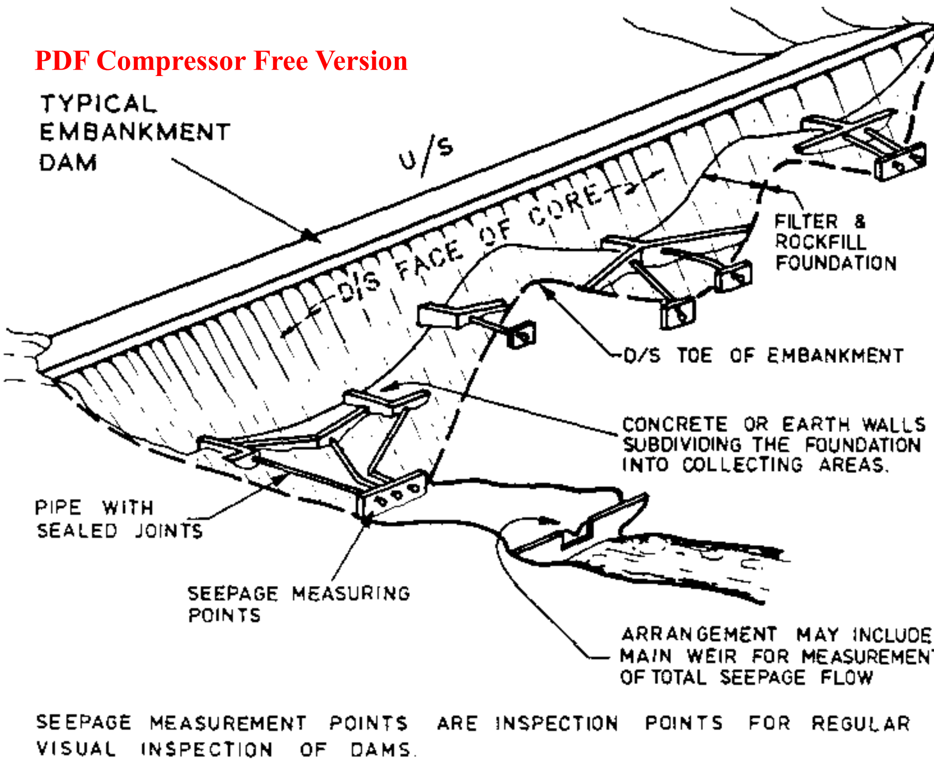


Figure 18.10. Seepage collection and measuring system for an earth and rockfill dam (ANCOLD 1983).

Chemical analysis of seepage can be a useful guide to the source of the seepage water, e.g:

– a comparison of ions in the reservoir water and seepage may indicate leaching of cement from grout curtains,

– biological analysis can indicate the source relative to depth in the reservoir,

– the age of water as determined by analysis of tritium can indicate its sources as rainwater or groundwater.

More importantly, routine inspection of seepage discharge should be made noting any discolouration of the water which may indicate piping of the embankment or the foundation. Any inexplicable increase in suspended solids, particularly during first filling, needs to be treated with some urgency as piping can develop rapidly.

The USA National Committee report in ICOLD (1989) refers to the monitoring of streaming potential and temperature in the dam foundations to monitor seepage. Variations from seasonal patterns are used to indicate changes in seepage conditions.

18.4.4 *Surface displacements*

Regular accurate survey of displacements of the surface of the dam embankment can be a most useful guide to performance.

Such measurements can be useful as a check on design assumptions, e.g. deformations of a CFRD, and as an indication of developing problems.

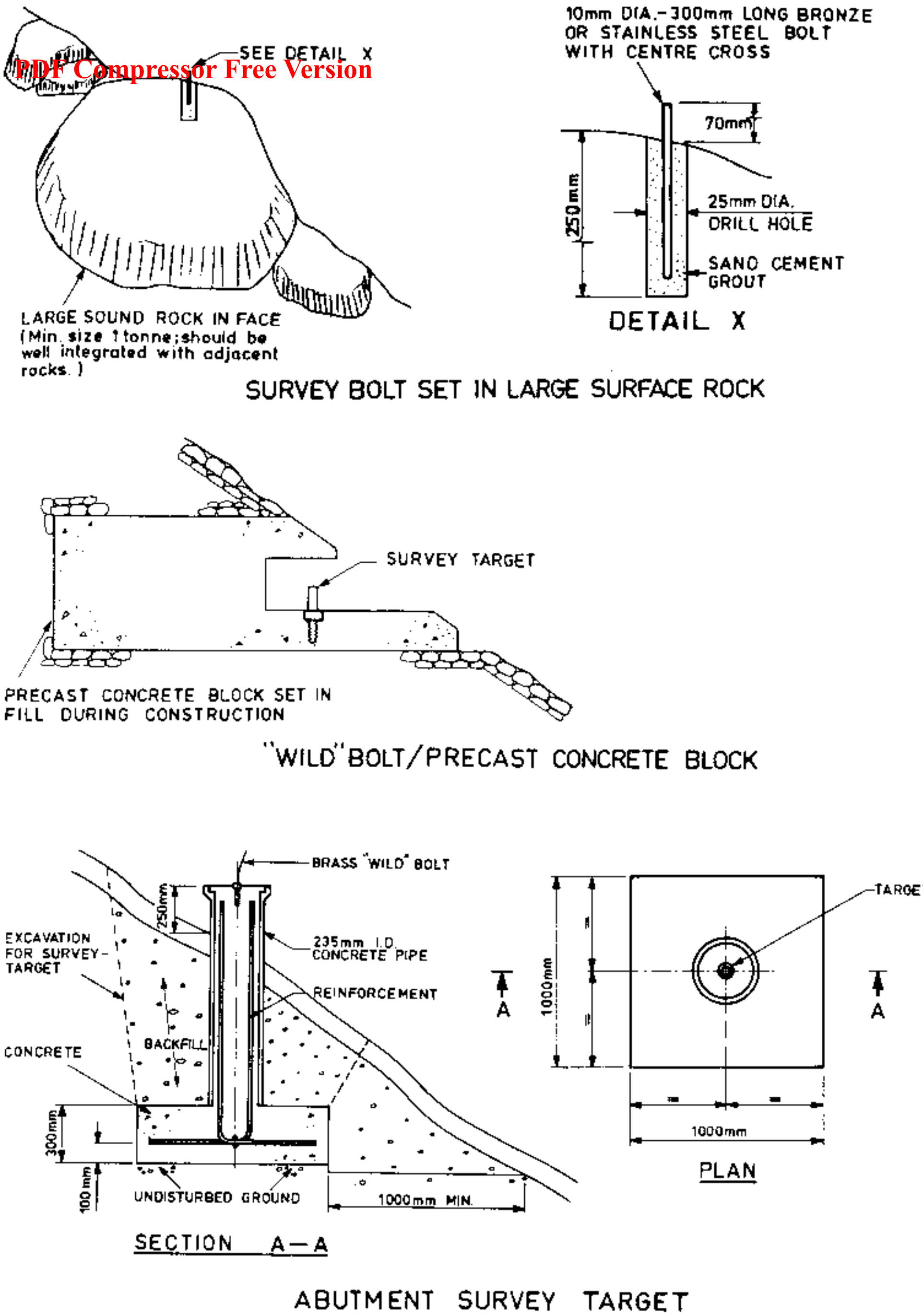


Figure 18.11. Surface settlement points for dams (ANCOLD 1983).

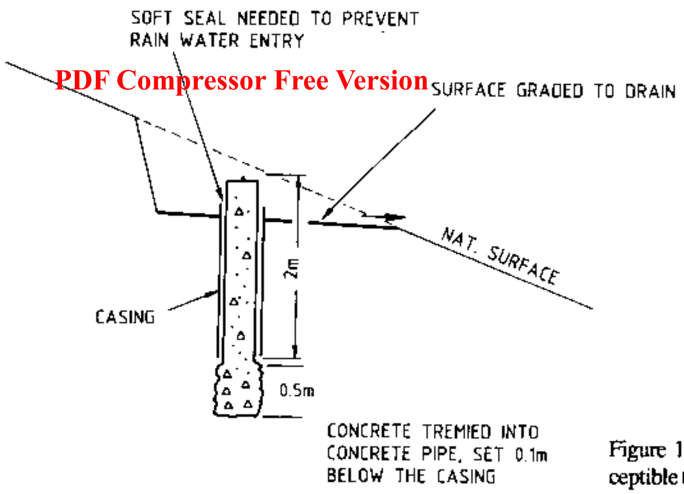


Figure 18.12. Survey point in soils susceptible to shrink and swell.

Figure 18.11 shows design of surface settlement points suitable for dams.

It is useful to extend the survey to surrounding areas, particularly if they are affected by slope instability.

In almost all cases, horizontal as well as vertical displacements should be measured, so that movement vectors can be determined. Such vectors can often give a good indication of the mechanism causing the displacement.

For monitoring of soil landslides or earthfill dams, it will be necessary to embed the survey points below the depth of shrink and swell due to seasonal moisture change. This may involve a system as shown in Figure 18.12, with the base at 2 to 4 m below ground surface, and the pillar isolated from the upper soil by a casing.

ICOLD (1988b) indicate that survey of displacements is falling out of favour. The authors' opinion is that this is unfortunate, and seemingly unnecessary, given the availability of quick accurate electronic survey methods.

18.4.5 *Pore pressures*

The measurement of pore pressures in the embankment and foundations of a dam can give vital quantitative information for use in assessing stability.

The following discussion gives an overview on why and where pore pressures are measured, the types of instruments available, their characteristics and some practical factors. For more detailed information on instuments, the reader is referred to Hanna (1985) and to manufacturers of the instruments.

18.4.5.1 *Why and where are pore pressures measured?*

There are a wide range of situations where it is necessary to measure pore pressures. The following gives examples to highlight some of the principles involved in locating the piezometers to obtain meaningful answers.

Dam embankment. Figure 18.13 shows piezometric conditions in an earthfill dam, constructed on an alluvial foundation, consisting of clay over sand. The ratio of horizontal to vertical permeability (k_H/k_v) of the earthfill is 15:1. Pore pressures in the embankment and foundations

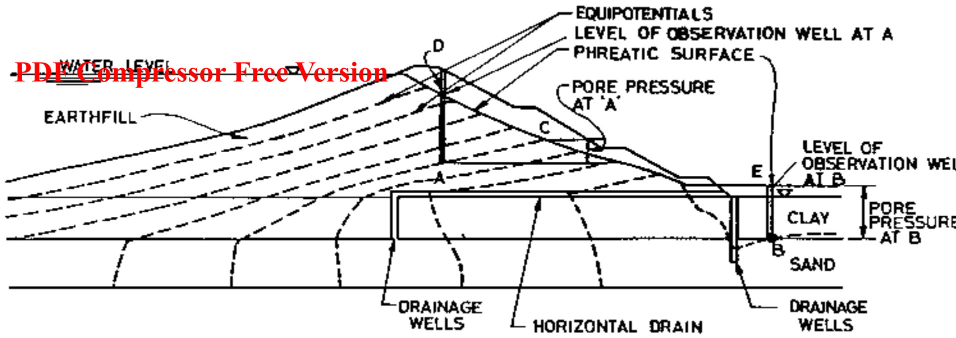


Figure 18.13. Embankment dam piezometric conditions.

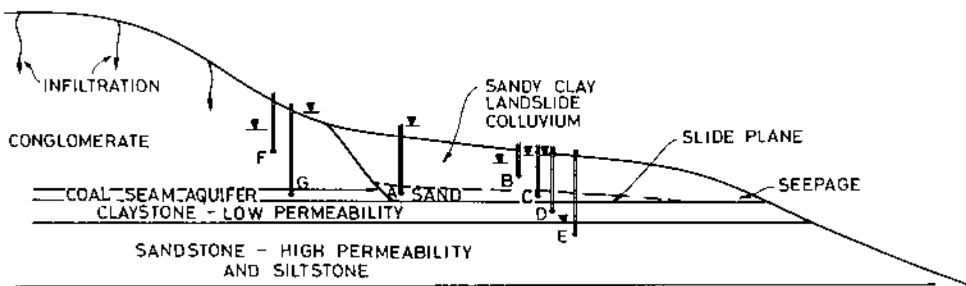


Figure 18.14. Landslide piezometric conditions.

are important from slope stability considerations, and for the potential for a 'blowout' condition to form at the toe of the embankment. Piezometers may be installed to check and monitor design assumptions.

The following points should be noted:

– An observation well, installed at A, with a sand surround and/or slots for its full length, will reach equilibrium at the phreatic surface at D.

– The pore pressure at A, i.e. the pressure that would be measured by a piezometer tip set at point A, is equal to the difference in elevation of A and the point where the equipotential through A meets the phreatic surface at C. Note that this may be much less than the difference in elevation from A to D, due to head losses in seepage flow through the earthfill.

– A slotted standpipe at B (with pipe extending above ground surface) will rise to the phreatic surface at E. In this case the pore pressure at B equals the head BE, because the pore pressure at B is higher than at any other point intersected by the standpipe. If the standpipe was cut off at the ground surface, water would flow from the pipe.

– It is the pore pressure in the embankment which is critical to stability, not the position of the phreatic surface. If k_h/k_v was only say 4, the equipotentials for the embankment would be higher for the same phreatic surface. Hence, pore pressures, not the location of the phreatic surface should be measured.

Landslide. Figure 18.14 shows piezometric conditions in a landslide. Steady state piezometric levels are shown for piezometers installed at points A to G.

The following points should be noted:

– The factor of safety against sliding is very sensitive to the pore pressures. From a slope stability viewpoint it is the pore pressure on the slide plane which is critical, i.e. at A and C. Piezometers (B) in the landslide colluvium may give different pore pressures.

– Piezometers (D), installed in a borehole drilled into a low permeability layer below the slide colluvium, should be backfilled with sand past the slide plane, or the response time for the piezometer will be very slow and pressures different to that on the plane may be measured.

– If boreholes are extended into a more permeable layer as for piezometer E, low pore pressures may be measured. If the sand backfill is extended to the slide plane, water may drain towards the sandstone/siltstone, and give lower pressures than actually are present at the slide plane.

– Piezometers installed in the low permeability conglomerate may take a long time to reach equilibrium and are likely to show lower piezometric levels than piezometers installed in the coal seam aquifer.

Jointed and sheared rock. Figure 18.15 shows a jointed and sheared rock mass into which a cutting has been excavated. The low permeability shear zone and clay infilled joint affect the flow of groundwater towards the excavation.

The stability of the cutting is determined by the strength of the joints, bedding planes and shear zone, which in turn is determined by the water pressure on these features.

If piezometers are installed in boreholes BH1 and BH2 as shown:

– Piezometer PX will read pressure due to phreatic surface C.
– Piezometer PY will read pressure due to phreatic surface B.
– Piezometer PZ will read pressure due to phreatic surface A.

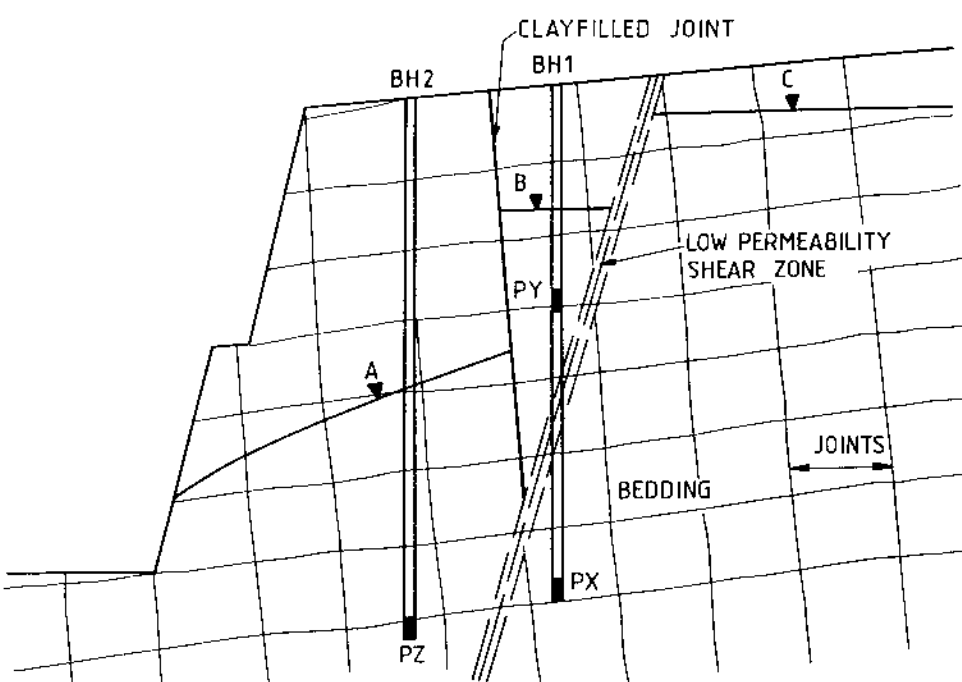


Figure 18.15. Piezometric conditions in jointed and sheared rock.

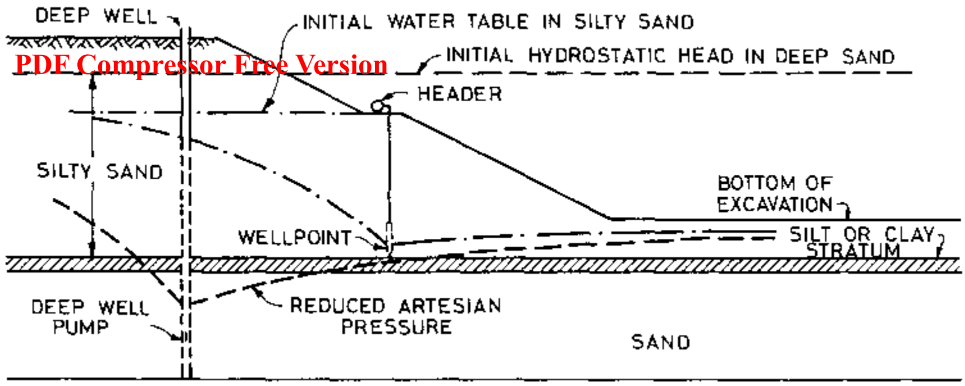


Figure 18.16. Dewatering an excavation.

In practice most rock masses are very complex, and it is not possible to simplify the model to the extent shown in Figure 18.15.

The geological factors which control groundwater movement must, therefore, be recognised when installing piezometers and several piezometers installed so that the critical condition can be defined. The piezometers should also be installed with a permeable surround, which intercepts the joints and bedding planes along which groundwater flows.

Dewatering an excavation. Figure 18.16 shows an alluvial soil profile into which an excavation has been constructed. The groundwater has been lowered by well points and deep well pumps.

Monitoring of the effectiveness of the dewatering system, by installation of piezometers, would be needed to check overall slope stability and heave of the bottom of the excavation. To achieve this, piezometers would have to be installed in the silty sand and in the sand because the pore pressure regimes are different. Several piezometers would be needed in each soil unit, because of the rapidly varying pressures away from the well points.

18.4.5.2 *Pore air and pore water pressure*

The effective stress in the soil σ' is given by

$$\sigma' = \sigma - u$$

where u = pore pressure.

In partially saturated soils the pore pressure (u) is a function of the pore air pressure (u_a) and pore water pressure (u_w)

$$u = u_a - \chi (u_a - u_w)$$

where χ is the fraction of the soil section that passes through water. For fully saturated soils χ = 1.0, and in a dry soil χ = 0.

In partially saturated soil, pore air pressure is commonly greater than pore water pressure due to surface tension effects on the air-water meniscus. Hence, when measuring pore pressures in partially saturated soils, the piezometers must be constructed so that either pore air or pore water pressure is measured. This is achieved, in most cases, by using high air entry filters for the piezometer element which measures pore water pressure, and low air entry filter where pore air pressure is to be measured. These are constructed of porous stone, ceramic or bronze. The pores are very small and uniform in size, and when saturated with air free water, air cannot pass into the filter until the applied air pressure exceeds the pressure developed on air-water meniscus at the entrance to each pore in the filter. Hence, if fine 'high air entry' filters are used, the piezometer will measure pore water pressure, and if coarse filters are used, pore air pressure will be measured.

Air and other gases may also be dissolved in the pore water, so even in saturated soils, air may enter the piezometer tip if it is too coarse.

In such cases, the air can lead to incorrect readings by lengthening the response time of the piezometer, and forming air-water menisci in the tubes, and standpipes which affect the measured pressure.

Sherard (1981) includes a discussion of the differences between pore air and pore water pressure in dam construction. He concludes that:

– In granular cohesionless soils $u_a \approx u_w$.

– In clay soils $u_a > u_w$, but not by more than 2 to 3 m of water head where $u_w > 5$ m water head. At higher pressures the difference rapidly decreases.

– The difference is always highest during construction when the soil is partially saturated.

Sherard quotes case histories where low air entry and high air entry tips have been used adjacent each other, with only small differences in readings. However, this experience is in large dams where pore pressures are large in magnitude. In smaller embankments the differences may be significant.

Sherard (1981) recommends use of high air entry tips for both hydraulic and vibrating wire piezometers. He suggests use of tips with an entry pressure of 500 to 600 kPa where these are available. He recommends use of tips with lower air entry pressures for pneumatic piezometers, the entry pressure being selected to be just greater than the anticipated soil suction (in partially saturated soils). This prevents transfer of water to the surrounding soil from the piezometer tip, while still allowing the transfer of the small volume of water required to operate the piezometer.

18.4.5.3 *Fluctuations of pore pressure with time and the lag in response of instruments*

In most practical situations the pore pressure being measured varies with time, e.g:

– pore pressures in a landslide will vary with rainfall,

– pore pressures around a pump well will vary with rate of pumping and the time since pumping began,

– pore pressures under an embankment on soft clay will vary with the height of the embankment and degree of dissipation by consolidation of pore pressures induced by the embankment,

– dynamic loading, e.g. earthquakes, may induce an increase in pore pressure.

In all cases, the piezometer which is selected should be constructed so that the time taken for the piezometer to respond to the change in pore pressure is sufficiently short to give a meaningful measure of the actual pore pressure.

In a simple standpipe piezometer, the piezometer responds to a change of pore pressure by flow of water into or out of the piezometer. This is a function of the geometry of the piezometer, the permeability of the soil and the change in pore pressure.

The response is discussed in Hanna (1973, 1985) and Brand & Premchitt (1980a, b).

Table 18.6 is reproduced from SAA (1981) and gives times for 95 and 99% equalization of Casagrande type piezometers with a 73 mm 'piezometer tip' and 9.5 mm inside dia standpipe.

Because of the small volume of water flow required to activate hydraulic, pneumatic,

Table 18.6. 95% and 99% equalization time lag for Casagrande Piezometers (SAA 1981).

Material	Sand	Silt			Clay		
Permeability cm/s	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}
Average time lag for 95% equalization	12 s	2 min	20 min	3.5 h	36 h	14 days	150 days
Average time lag for 99% equalization	18 s	3 min	30 min	5.2 h	54 h	21 days	225 days

vibrating wire and strain gauge piezometers, their time lag is very small compared to Casagrande piezometers.

For hydraulic piezometers, the 'response' time depends on whether the piezometers have to be flushed, in which case time has to be allowed for pressures to reestablish equilibrium due to the flushing affecting pore pressures around the piezometer tip. It also depends whether individual pressure gauges are provided for each piezometer, or whether a master gauge is used. In the latter case, time must be allowed for equilibrium to be achieved.

Pneumatic piezometers have a small but finite response time, as a small displacement of the diaphragm is needed. The response time varies depending on whether high or low air entry tips are used.

Vibrating wire and strain gauge piezometers have virtually zero response time as there is no transfer of water involved.

18.4.5.4 *Types of instruments and their characteristics*

Observation well. As shown in Figure 18.17 an observation well, or 'open standpipe,' consists of a slotted plastic, reinforced fibreglass or steel pipe attached to a standpipe, installed in a borehole, and surrounded with sand. Porous tips, as described below for Casagrande piezometers, may be used. When slotted pipe is used, it is often wrapped in filter fabric to prevent clogging by the sand. The hole is sealed at the top with mortar to prevent surface water flowing in. The water level in the well is measured with a water level dip meter.

An observation well normally measures the maximum water pressure intersected in the hole. It does not measure pressure at a point, and does not account for flownet effects (see Fig. 18.13. In some situations, with confined aquifers of varying permeability, an observation well may not measure the maximum pressure, with water from the high pressure zone flowing into a more permeable lower pressure zone.

The time lag for observation wells may also be large, since a relatively large flow of water is required for the well to reach equilibrium with any pressure change.

Casagrande piezometer. Figure 18.18 shows a Casagrande type piezometer. These are a development of the open standpipe and consist of a porous tip which is embedded in a sand filter, and sealed into a borehole with a short length of bentonite. The hole is usually backfilled with cement grout, or the soil excavated from the hole. The standpipe is kept as small a diameter as practical to keep the response time to a minimum. Dunnicliff (1982) recommends the standpipe be not smaller than 10 mm dia so that gas bubbles will not be trapped. In any case, this is a practical minimum for use of a dip meter to measure the water level. The sand filter acts as a large collector of water compared to the small diameter standpipe. Vaughan (1969) examined the problems of sealing a piezometer in a borehole, and gives formulae to determine the potential error due to inadequate sealing. In practice a bentonite seal 0.5, or preferably 1.0 m thick, is sufficient for most cases.

The bentonite seal may be formed using dry pellets which are available commercially, or by

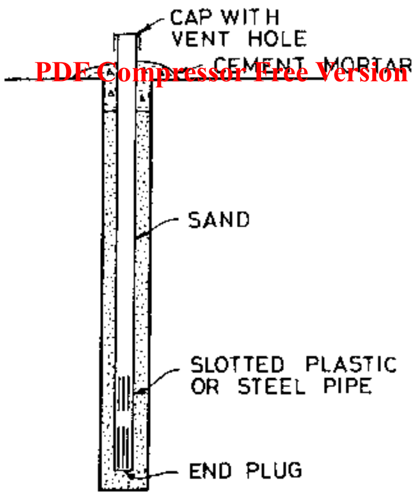


Figure 18.17. Observation well.

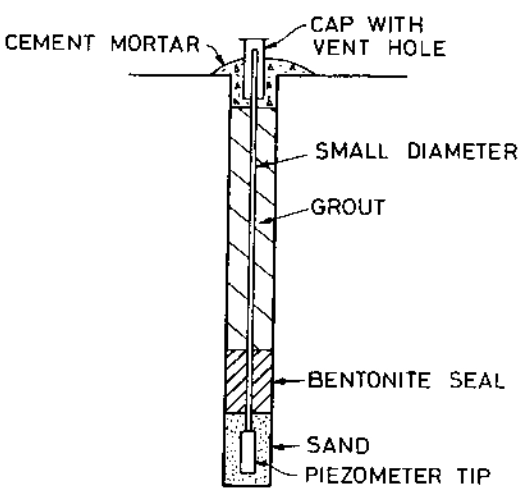


Figure 18.18. Casagrande type piezometer.

making 25 mm diameter moist balls of powder bentonite. These are dropped down the hole, possibly tamped to compact and allow checking of the position and depth of the seal.

Casagrande, and all standpipe piezometers, measure pore water pressure (u_w). They are potentially accurate, depending on how carefully the water level is determined. The time lag can be determined as outlined in Section 18.4.5.3. Time lag may be a problem in lower permeability soils, and where the pore pressure is changing rapidly.

The Casagrande type piezometer is low cost, simple to construct with readily available equipment and materials. They are self de-airing, measure pore water pressure and have a long performance record. Disadvantages include susceptibility to damage by construction equipment, animals and by consolidation of soil around the standpipe, susceptibility to freezing, inability to measure pressures higher than the level of the standpipe, or pressures lower than atmospheric. If porous filter tips are used, these may clog with chemical deposits with flow in and out of the tip. Reading is simple but relatively time consuming. Auto-data logging is not practicable.

Hydraulic twin tube piezometer. Figure 18.19a shows an Imperial College (Bishop) type hydraulic tube piezometer. Variations of the principle include the USBR type shown in Figure 18.19b. These consist of a porous tip, which is usually installed during construction (e.g. in a dam embankment), from which two tubes are led to a convenient terminal measuring point where gauges are used to record the pressure. The total pressure equals tip pressure, plus elevation difference from the tip to the measuring gauge.

Two tubes are provided so that air (gas), which is trapped in the tubes or which enters through the porous tip, can be flushed from the system using de-aired water. The tips are constructed of fine porous ceramic, stone, aluminium or sintered bronze. The tips are required to be strong enough to withstand the total pressure, and fine enough to prevent clogging by the soil. More importantly, the tips must be 'high air entry' to limit entry of air into the system. Commonly, high air entry tips can withstand an air pressure of 100 to 200 kPa, but may be up to 500 to 600 kPa.

Coarse, low air entry tips are not used with hydraulic piezometers because they allow air to enter the tip, the piezometer and tubes. This leads to pore air pressure rather than pore water pressure being measured in partially saturated soils, a longer response time for the piezometer, and incorrect elevation pressure adjustment if the air enters the tubes. The high air entry tips are only effective if they are completely de-aired before installation. This is usually done by boiling for 2 to 6 hours. Sherard (1981) suggests 12 to 24 hours boiling.

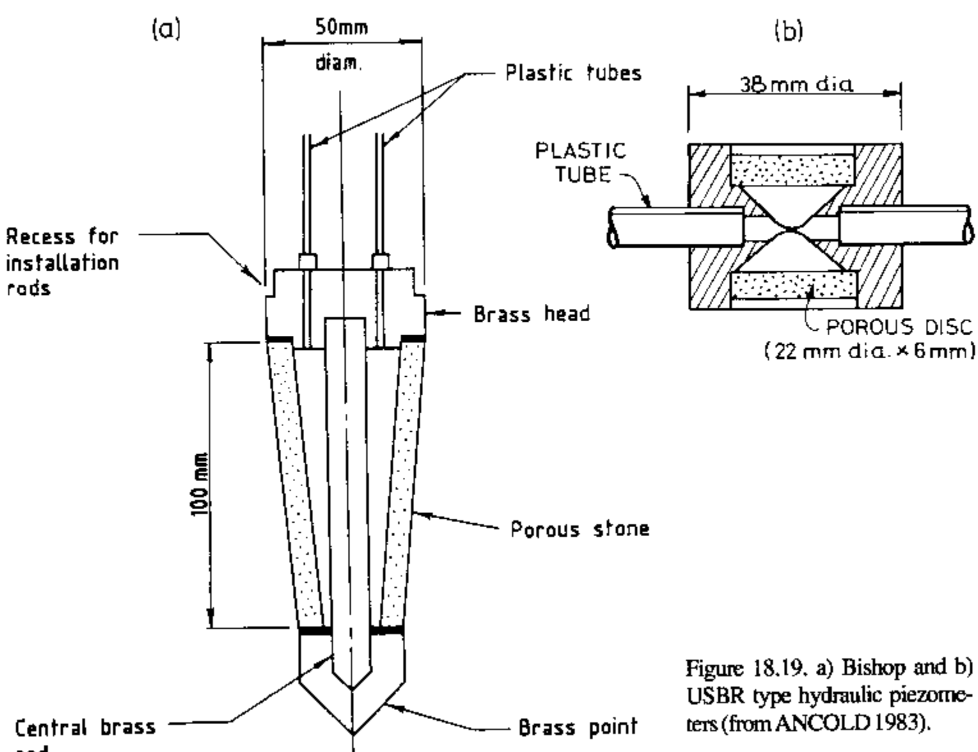


Figure 18.19. a) Bishop and b) USBR type hydraulic piezometers (from ANCOLD 1983).

The tubes leading to the measuring gauge should be constructed of Nylon 11, covered by polyethylene. The Nylon 11 is impervious to air, the polyethylene impervious to water, and the combination is impervious, strong, flexible and not affected by chemical attack.

Despite the use of high air entry tips, and the proper tubing, gas may still accumulate in the tubes, and periodic flushing with de-aired water is necessary. Some organisations add bacteriacides and algacides to the water, to prevent algae growth blocking the tubes.

Some organisations use flow meters on the inlet and outlet lines as an aid to flushing. A flow of 300 ml/min is maintained to prevent air becoming trapped in high spots in the lines. It is also good practice to flush before all readings, since equal pressures on the lines can occur with air in both lines.

Eagles (1987) reports that if the entry to the piezometer is smaller than the tubing (e.g 1.5 to 2 mm compared to 5 mm), crystal growth from chemicals dissolved from the tubing can cause blockage, and it is important that the entry be the same diameter as the tubing.

The tip pressure is measured using Bourdon gauges, measuring manometers, or pressure transducers in the terminal structure. Most installations use Bourdon pressure gauges – either one gauge for each piezometer, with a master gauge to check, or a master gauge with a switching system. Typical layouts are given in ANCOLD (1983). When using a master gauge, it is necessary to estimate the tip pressure and pressurize the gauge to this pressure before connecting the piezometer. Any imbalance will involve flow of water into or out of the piezometer tip, with resultant lag time, and possibly influence the pressure, particularly if the soil is partially saturated, and water flows out of the piezometer tip. Use of a gauge for each piezometer with a check master gauge overcomes this.

The terminal structure and tubes must be located so that no more than about 5 m negative head is generated, or cavitation will occur in the tubes, giving incorrect readings.

In embankment construction, the tubes have to be laid in trenches to avoid damage by construction equipment. Trenches are usually filled with sand, with bentonite or clay cutoffs in the trenches to prevent water seepage along the trenches.

Provided that high air entry tips are used, and the tips installed with intimate contact with the soil, hydraulic piezometers will read pore water pressure. Subatmospheric pressures up to about 50 to 70 kPa can be read.

Twin tube hydraulic piezometers are relatively simple and medium cost to construct, although when the cost of terminal house (which may have to be dewatered to position it satisfactorily, to avoid excessive negative pressures) is included, the overall cost may be comparable with vibrating wire and pneumatic systems.

Auto-data logging is possible by using electrical pressure transducers. Hydraulic piezometers have a good long term reliability (15 to 20 years in many cases), and have been the favoured instruments in British and Australian dams. They were used in USBR dams until 1978 when pneumatic piezometers were introduced, because they are easier to read and maintain in good working condition. Disadvantages include 'growths' in the tubes; damage of tubes by differential settlement; the need for an elaborate terminal house, often requiring dewatering; skill and care to keep them operating, and the need to avoid high subatmospheric pressures in the lines. Particular advantages are the ability to flush air from the piezometer tip if necessary, hence, ensuring measurement of u_w, ability to measure moderate subatmospheric pressures, and the possible use of the piezometer to carry out *in situ* permeability and hydraulic fracture tests of the soil. As pointed out by Sherard (1981), hydraulic (and pneumatic and vibrating wire) piezometers should not be installed in a surround of sand: Doing so only slows the response time if the sand surround becomes partially saturated, and since the piezometers are essentially no flow, a large sand surround has no benefit in collecting water.

Pneumatic piezometer. Figure 18.20 shows the principle of a pneumatic piezometer. These consist of a piezometer tip, from which two tubes are led to a convenient terminal measuring point. Pressure is measured by applying pressurised gas (usually dry nitrogen) on the inlet tube. The diaphragm is moved, and gas escapes to the outlet tube where it can be observed. The pressure on the inlet tube is then reduced until the diaphragm closes (no flow from the outlet tube). The pressure at which the diaphragm closes is the pore pressure (after correction for any closure spring effect). Some manufacturers measure the pressure on opening of the diaphragm but most use the principle outlined above.

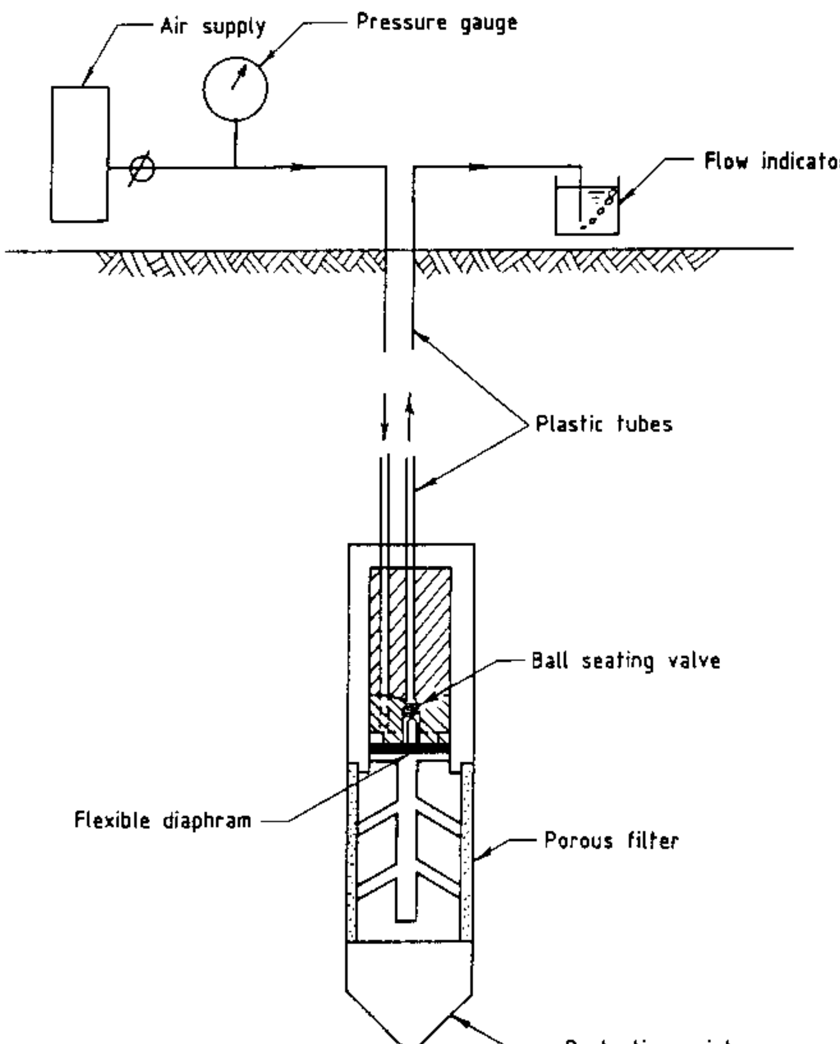


Figure 18.20. Pneumatic piezometer (from ANCOLD 1983).

The piezometers are fitted with high or low air entry tips. Sherard (1981) discusses at some length the relative merits of using high or low air entry tips. He concludes that if low air entry tips are used in unsaturated soil, the water in the tip will be sucked out by capillary action, and the piezometer will measure pore air pressure u_a. However, if high air entry tips are used, the response time during reading may be affected, and the piezometer may read an erroneously high pressure. This is because of the small volume of water displaced by the action of the diaphragm. Sherard suggests using tips with an air entry value just above the soil suction pressure. This is not a problem when the piezometers are being used in saturated soils, and since it is easier to de-air low air entry tips, these are often used. Where high pore pressures are to be measured the pore air and pore water pressures are similar so the choice of tip is not critical.

The tubes should be of the same construction as for hydraulic piezometers, i.e. Nylon 11 sheathed in polyethylene, as this gives strong, stiff tubes, impervious to ingress of water and leakage of air. If water does enter the tubes, flushing with dry nitrogen is necessary. The use of lighter tubing can result in problems with crushing and kinking, and the savings in cost may not be warranted.

The pressure is usually read by a portable readout which is connected to the tubes with quick-connect couplings. Either a Bourdon or digital type gauge is used. The latter can be adapted to auto-data logging with the terminal permanently installed and programmed to interrogate each gauge in turn at predetermined intervals.

The terminal pressure gauge can be located at any elevation relative to the piezometer and tubes can be laid in any configuration. In embankments, the tubes must be laid in trenches to avoid damage by construction equipment and looped, coiled etc. to allow for differential movement.

Because of their method of construction and reading, pneumatic piezometers cannot be used to measure subatmospheric pressures.

Pneumatic piezometers are relatively simple and low cost construction, and the terminal gauge house requirement is much simpler than for hydraulic piezometers. Pneumatic piezometers have performed satisfactorily over 10 to 15 years in the USA (Sherard 1981) and are now the preferred instrument for dams in that country because they are easier to read, easier to install and auto-data logging is simple. Less skilled operation is needed than for hydraulic piezometers. There has been some reticence to adopt them for dams in some other countries, including Australia, but that seems to be changing. They have been used in boreholes in rock (and in concrete dams) by installing with a packer, allowing replacement if this was necessary. The time for taking measurements is dependent on the readout equipment and the length of the tubes to the piezometer. Sherard (1981) quotes read times of 1 to 2 minutes for short tubes, but up to 10 to 20 minutes for very long tubes. He indicates this was reduced to 3 to 5 minutes by initial rapid gas filling. There are no problems of freezing with pneumatic piezometers.

Disadvantages include long measurement time for long tubes, and high and subatmospheric pressures cannot be read.

Vibrating wire piezometer. Figure 18.21 shows the principle of a vibrating wire piezometer (also known as an acoustic piezometer). This consist of a piezometer tip from which an electrical cable leads to a convenient terminal measuring point. The piezometer consists of a porous tip, as in a pneumatic piezometer with a stiff metallic diaphragm. The diaphragm is attached to a prestressed wire. When the diaphragm deflects with changes in pore pressure, the tension in the wire changes. The natural frequency of the wire is dependent on the tension. The natural frequency is measured by plucking the wire using an electromagnet. The wire then vibrates in

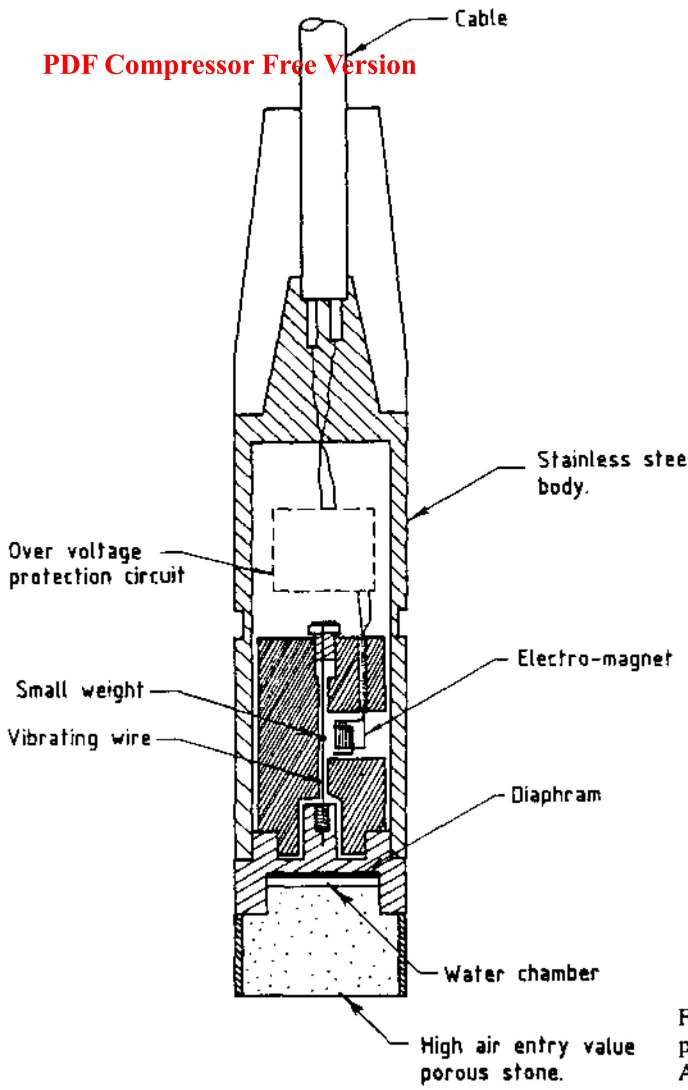


Figure 18.21. Vibrating wire piezometer, Maihak type (from ANCOLD 1983).

the magnetic field of a permanent magnet, causing an alternating voltage of the same frequency as the wire, and this can be measured at the measuring point.

Hence, by measuring the natural frequency of vibration of the wire, and calibrating this against pressure on the diaphragm, the instrument can be used to measure pore pressure.

The instruments are essentially a no flow devices and, therefore, have a very short response time. Sherard (1981) suggests that they are best fitted with high air entry tips to limit entry of air into the piezometer, and ensure that pore water pressure is measured. A digital readout is usually provided, and the instruments are readily suited to auto-data logging. The readout unit can be located at any elevation relative to the piezometer, and up to several kilometres away. The wires can be laid in any configuration, but provision should be made for differential movement.

Provided they are installed with intimate contact with the soil, vibrating wire piezometers can be used to measure subatmospheric pressure.

Vibrating wire piezometers are more expensive than hydraulic or pneumatic piezometers, but have been used widely in European, USA, Canadian and Australian dams, with reliable performance for up to 20 years (Sherard 1981). They are the easiest and quickest to read of all piezometers. They are not susceptible to freezing.

There are varying opinions on whether they are susceptible to long term drift, due to creep of the diaphragm or tension wire. Sherard (1981) presents information suggesting this is not a problem. Other advantages are that by using high air entry tips, the saturation of the tips can be checked by observing the development of negative pressure as the tip is allowed to dry slightly, and in partially saturated soils, water will not be sucked from the tip into the soil (unlike hydraulic piezometers which may wet the soil during the flushing operation).

Disadvantages have included damage by lightning strike, particularly during construction when the cables are exposed. This has been overcome by shielding cables, earthing, and provision of overvoltage protection. Stray currents from nearby power stations have also caused problems.

Strain gauge piezometer. Strain gauge piezometers are similar in construction to the vibrating wire piezometer, but the deflection of the diaphragm is measured by either bonded electrical strain gauges, or Carlson type unbonded strain meters. Pore pressures are recorded as changes in resistance. The instruments are not as widely used as vibrating wire piezometers, mainly because of the possibility of drift in the long term and with temperature. However, the instruments have the advantage of being easy to read, easily auto-data logged, independent of the relative location of the piezometer tip and readout unit. As with vibrating wire instruments, they are susceptible to damage by lightning strike, and overvoltage protection is needed.

18.4.6 *Internal displacements and deformations*

Internal displacements and deformations are often monitored on larger earth, earth and rockfill, and concrete face rockfill dams. This allows confirmation of design assumptions, particularly relating to long term settlement of earthfill and foundations, and to concrete face designs.

Hanna (1985) and ANCOLD (1983) give details of the instruments used. Up to date information can be obtained from instrument manufacturers. The following is a brief overview of the types of instruments used:

18.4.6.1 *Vertical displacements and deformation*

Vertical displacements and deformations are measured using mechanical or magnetic devices which are either embedded in the fill as the dam is constructed, or installed in boreholes. Figure 18.22 shows the principle of the USBR type mechanical device.

The base tube is installed in the dam foundation, and a cross arm anchor placed in the dam foundations. Telescoping tubing is placed in the fill as the dam is constructed, with cross arm anchors being installed at intervals to allow differential settlement within the fill to be measured. Typically cross arms will be spaced at 2 or 3 m intervals. Settlement readings are obtained by survey of the top of the tubing, and lowering a measuring torpedo down the tube. The torpedo is provided with spring loaded arms which engage the lower ends of the cross arm pipes. It is supported on a measuring tape, and can allow accuracy of measurement of 3 mm (accuracies as small as 1mm are claimed). Settlements as much as 15% can be accommodated, and the

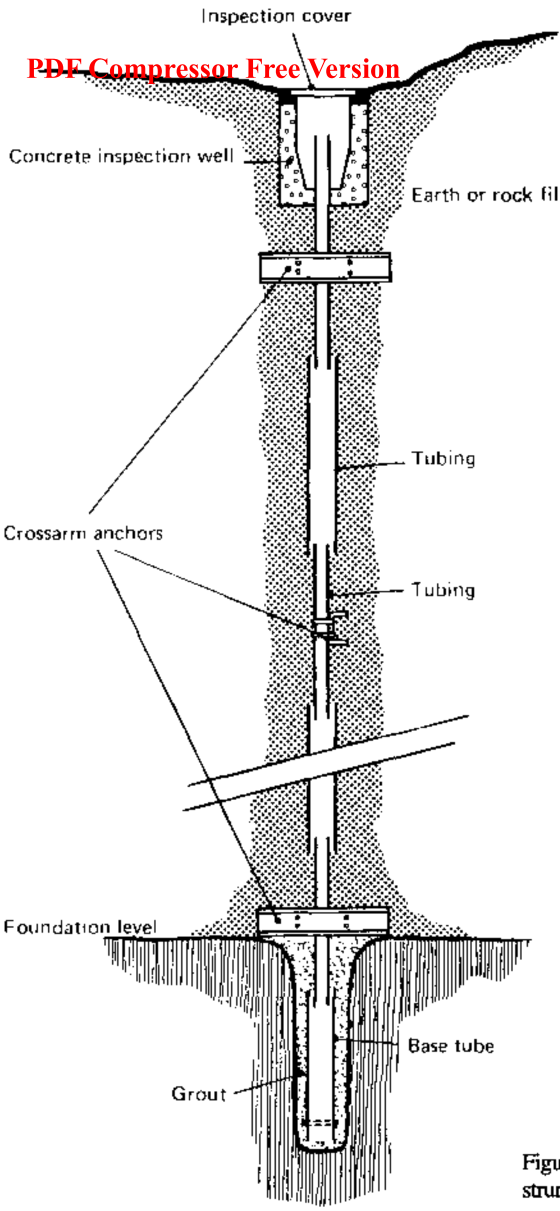


Figure 18.22. USBR settlement gauge (Soil Instruments Ltd 1985).

testing can be installed to depths of at least 100 m. The tubing does not have to be vertical (Soil Instruments Ltd 1985) indicate 25° from vertical is allowable.

The USBR instrument cannot be installed down a borehole, only in fill as it is constructed.

Figure 18.23 shows a magnetic version of the instrument which uses ring magnets instead of the crossarms.

In fill, ring magnets are placed in the ground at predetermined intervals, in the same way as the cross arms are for the USBR device. In boreholes (Figure 18.23b), a spring loaded magnet

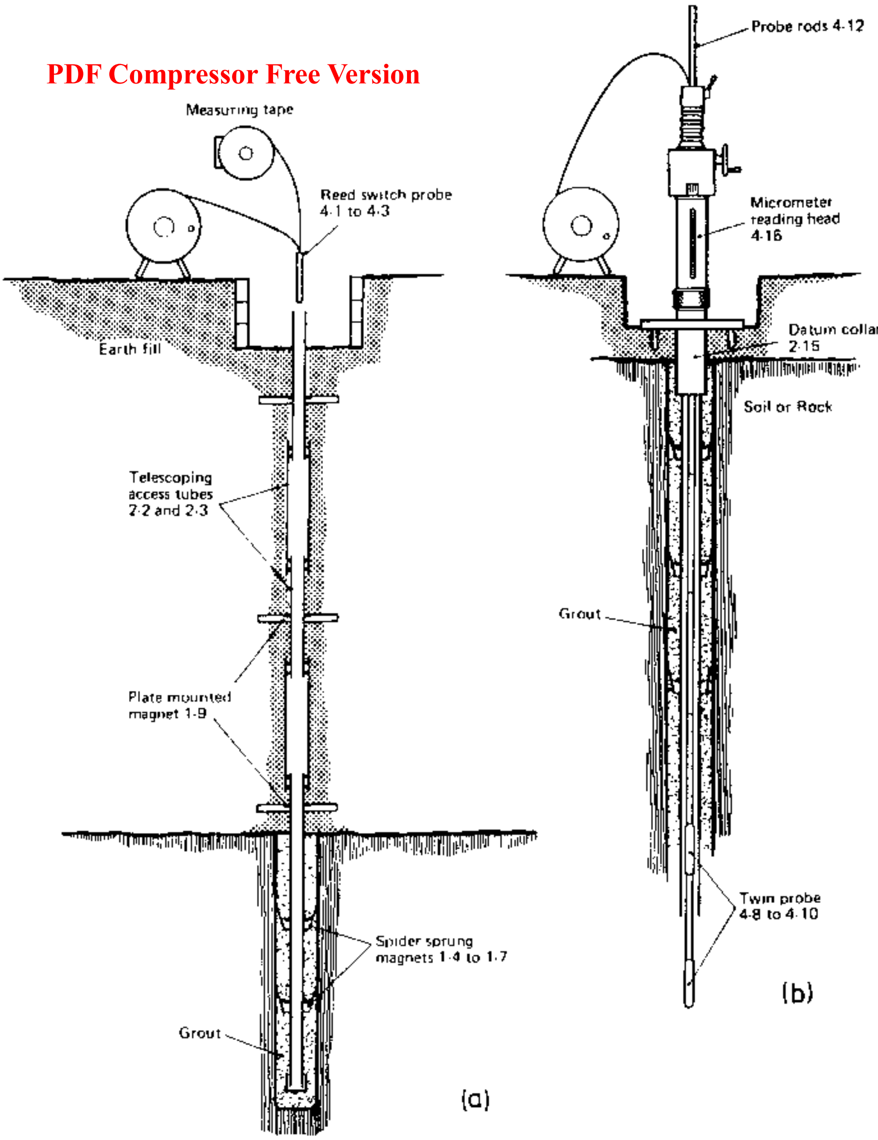


Figure 18.23. Magnet (Soil Instruments Ltd 1985) settlement gauge.

is lowered through the grout supporting the borehole, and is held to the walls of the hole by the spring and the grout. The grout has a compressibility equal to, or greater than, the soil or rock surrounding it. The probe is lowered down the tube and senses the position of the magnets. If suspended by a tape, the accuracy of measurement is 1 mm. Soil Instruments Ltd (1985) indicate that a rod mounted version fitted with a micrometer device to help position the magnet can read to 0.1 mm.

The tape suspended device can be used in the same conditions as the USBR device, i.e. in

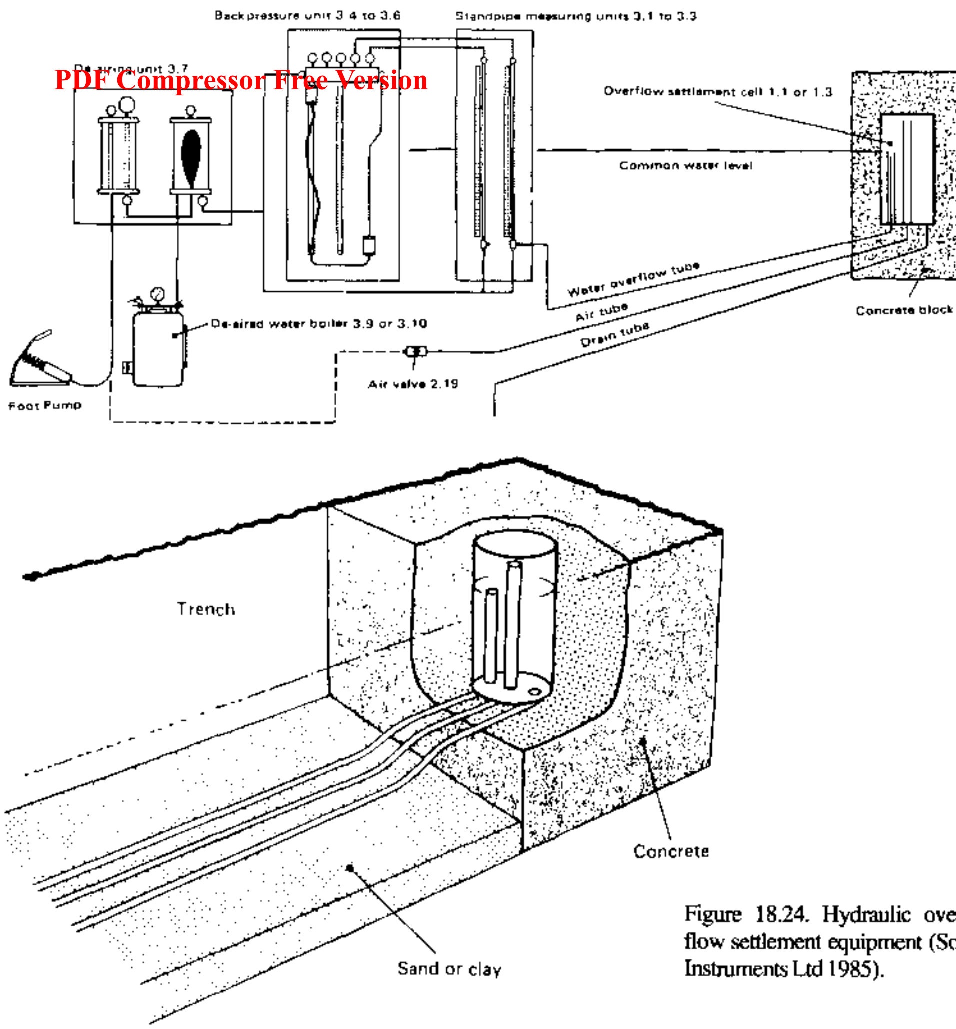


Figure 18.24. Hydraulic overflow settlement equipment (Soil Instruments Ltd 1985).

near vertical holes. The rod mounted device can be used in any orientation, so can be used to measure horizontal displacements, as well as vertical.

Figure 18.24 shows the principle of hydraulic overflow settlement cell.

These work on a simple U tube principle. The cell is cast into the fill, and connected by a water tube to a graduated standpipe. A second drain tube allows surplus water to flow from the cell and a third 'air' tube maintains the interior of the cell at atmospheric pressure. Compressed air is used to clear the air tube and drain tube, then the water tube is filled with de-aired water, pumping sufficient water to remove any bubbles in the water tube. After allowing time to reach equilibrium, the level of water in the standpipe is the same as that at the cell. The range of the instrument can be extended by applying small back pressures to the standpipe.

The method is very simple and inexpensive, and according to Soil Instruments Ltd gives an accuracy of the order of 1 to 5 mm, depending on the back pressure being applied. Hanna (1985) indicated an accuracy of 2 mm. Tube lengths up to 300 m can be used. ANCOLD (1983) gives details of such a device, which uses two cells at the same location. These are at slightly different levels, allowing checking of whether there is air in the lines, and rotational movement of the cell.

These devices are used in rockfill zones where it would be difficult to install cross arm devices, but may be used in earthfill. Hanna (1985) and Soil Instruments Ltd (1985) describe the use of pneumatic and vibrating wire versions of the hydraulic settlement cell.

18.4.6.2 *Horizontal displacements and deformations*

On some very large dams, horizontal displacements and deformations are measured using mechanical and electronic devices embedded in the dam fill as the dam is constructed. Their use is usually combined with hydraulic settlement gauges, so that the settled profile can be determined. The mechanical instruments are installed in rockfill zones, it being considered unwise to place them through the earth core as they may form a preferred leakage path.

Figure 18.25 shows a mechanical device which uses long wires, anchored to crossarms embedded in the dam. The wires are held at constant tension, and displacements measured at the terminal well which is situated on the face of the dam.

The wires are housed in telescoping segments of pipe. Corrections for overall translation is made by surveying the position of the terminal well. ANCOLD (1983) indicate the accuracy to be 1.5 mm.

Soil Instruments Ltd (1985) and Hanna (1985), describe other extensometers which can be used to measure horizontal displacements and strains. These include devices which use vibrating wire and resistance potentiometers to do the measurements.

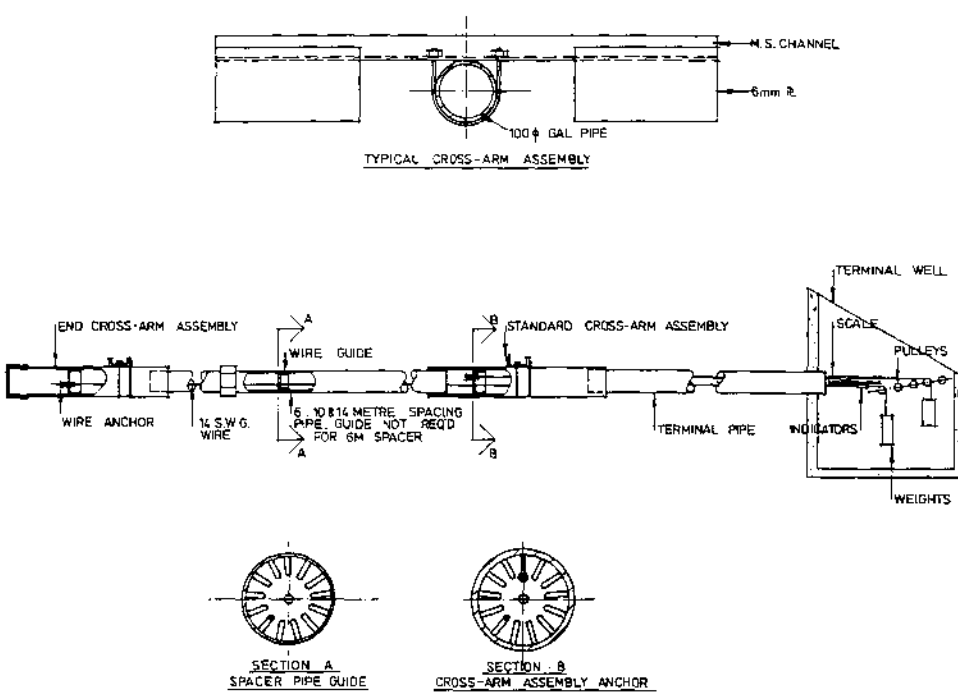


Figure 18.25. Long wire extensiometer (ANCOLD 1983).

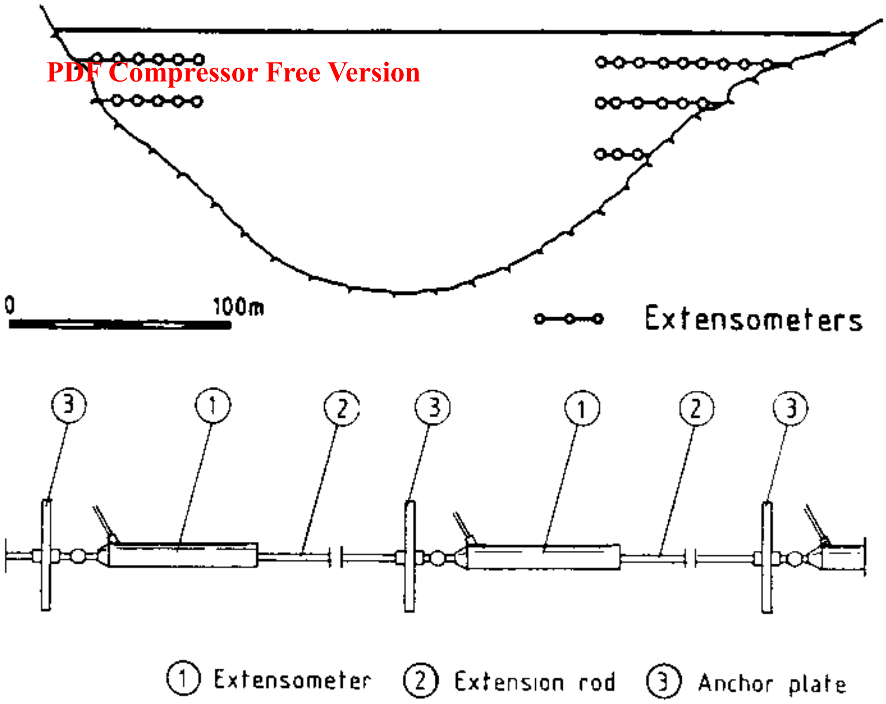


Figure 18.26. Use of extensometer to measure strain parallel to the dam axis (ICOLD 1989).

Figure 18.26 shows the use of such devices to measure deformations in the earth core of a dam parallel to the dam axis. In this case, several extensometers are installed with extension rods connecting the anchor plate and the vibrating wire extensometers.

Lateral movements in dams can also be measured by use of borehole inclinometers. The inclinometer casing may be installed during construction of the dam, or after, in the event that excessive displacements are observed. If the casing is installed on a slope, e.g. along the face slab of a concrete face rockfill dam, horizontal and vertical displacements can be determined.

Most inclinometers are similar to that shown in Figure 18.27, in that the inclinometer is passed down casing which is grooved to maintain the orientation of the sensor probe.

The tilt of the sensor is measured and recorded by servo-accelerometers located within the probe. Where two accelerometers are used they can sense the inclination of the tube in two directions at right angles to each other. Processing of the signal at the surface allows display of the angular rotation, or displacement normal to the casing in the two directions. Settlement gauge magnets may be installed around the casing to allow assessment of axial deformations.

The casing may be plastic or aluminium. The casing should be able to telescope, to allow for differential settlement. Soil Instruments Ltd (1985) indicate a resolution of 0.1 mm. Hanna (1985) indicates that careful calibration and maintenance is necessary to achieve a high precision.

Hanna (1985) indicates that the grooves in the casing may spiral, and individual lengths need to be checked for this.

ANCOLD (1983) describe an inclinometer which can be used in smooth pipe. It contains

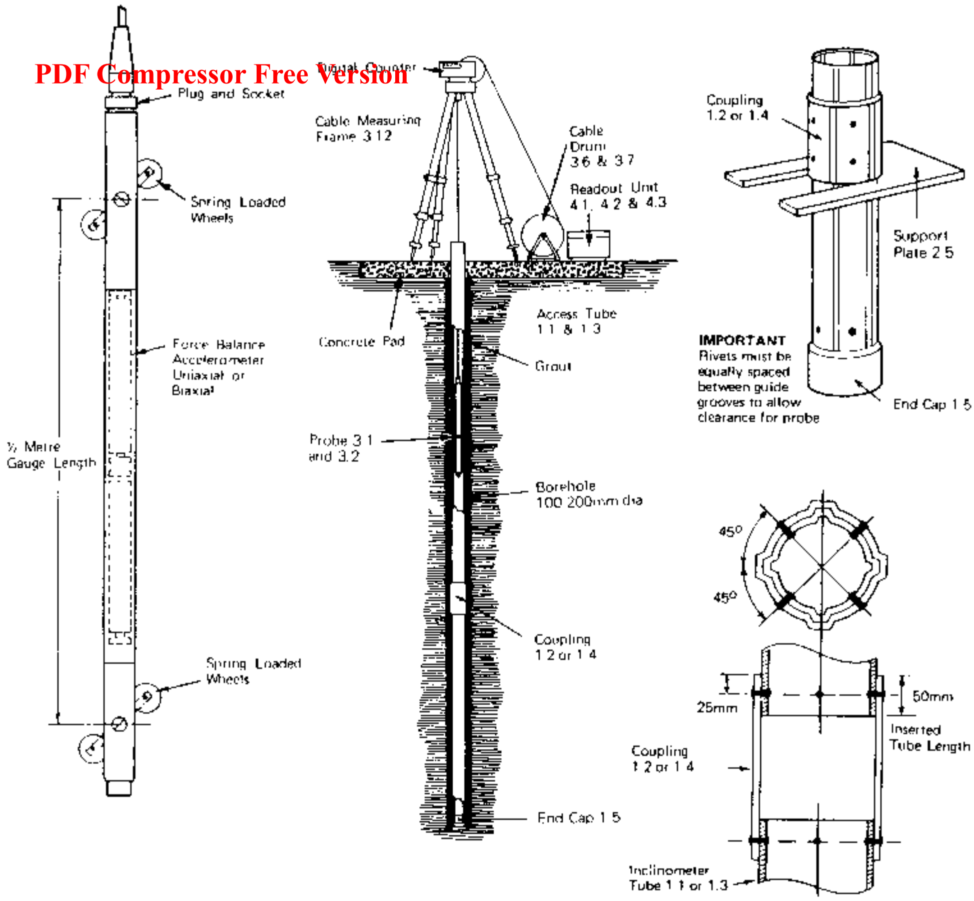


Figure 18.27. Borehole inclinometer (Soil Instruments Ltd 1985).

two servo-accelerometers. The device was developed for monitoring displacements of the face of concrete face rockfill dams. The accuracy is 3 mm.

18.5 HOW OFTEN SHOULD MONITORING BE CARRIED OUT ?

The frequency of inspection and measurement of instruments will depend on the size and hazard rating of the dam, whether the dam is filling for the first time or has been in operation for many years, whether deterioration has been observed etc.

Some guidance can be obtained from Tables 18.3a and b and from Tables 18.7 to 18.9.

Automation and telemetering of readings to a control monitoring point is becoming more common. However, in ICOLD (1989), several national committees (e.g. France, Switzerland) caution against over reliance on automation and, counsel the use of visual inspection as a vital part of dam monitoring.

Table 18.7. Instrument monitoring schedule (USA National Committee in ICOLD 1989).

Period	Type of measurement		
	Deflection/deformation	Stress/strain/temperature	Seepage/piezometric levels
During construction	PL – Read weekly SL – Read prior to filling FD – Read weekly MP – Read weekly	SS – Read weekly SM – Read weekly T – Read weekly	U – Read weekly D – Read weekly P – Read weekly
First filling	PL – Read daily during fill or each specified rise SL – Read once after reservoir reaches level to be maintained FD – Read daily during fill or each specified rise MP – Read daily during fill or each specified rise	SS – Read once each specified rise SM – Read once after reservoir reaches level to be maintained T – Read once after reservoir reaches level to be maintained	U – Read following filling D – Read following filling unless unanticipated flow is encountered P – Read daily during fill or once each specified rise
Initial holding (if applicable)	PL – Read daily for first week, weekly thereafter SL – Read monthly FD – Read weekly MP – Read weekly unless creep is indicated	SS – Read weekly SM – Read weekly T – Read weekly	U – Read daily for first week, weekly thereafter D – Read weekly P – Read daily for first week, weekly thereafter
Subsequent first year's operation	PL – Read bi-monthly SL – Read quarterly FD – Read monthly MP – Read monthly	SS – Read bi-monthly SM – Read bi-monthly T – Read bi-monthly	U – Read weekly D – Read weekly P – Read weekly
After dam attains stabilized pattern of behavior	PL – Read monthly SL – Once a year at high reservoir FD – Read montly MP – Read monthly	SS – Read monthly SM – Read monthly T – Read monthly	U – Read weekly D – Read weekly P – Read weekly

PL = plumblines, SS = stressmeters, U = uplift pressure, SL = survey transverse, triangulation, SM = strainmeters, D = seepage, FD = foundation deformation meters, T = thermometers, P = Piezometers, MP = multiple position extensometers.

Table 18.8 Monitoring for Revelstoke Dam (Canadian National Committee in ICOLD 1989).

Stage	Instruments									
	Core piezometer	Foundation and shell piezometer	Vertical movement gauge	Horizontal movement gauge	Horizontal strain gauge	Surface monument	Earth pressure cell	Weir and well	Strong motion accelerograph	Visual inspection
During construction	Frequently by field staff				1/month	1/month	1/month	1/month	Continuous	1/month
Reservoir filling	1/2 days	1/2 days	1/week	2/month	2/month	1/month	2/month	1/2 days	Continuous	1/day
After the first reservoir filling										
First 6 month	1/week	1/week	1/month	1/month	1/month	1/month	1/month	1/week	Continuous	1/day
6 month to 1½ year	2/month	2/month	4/year	4/year	4/year	4/year	4/year	2/month	Continuous	1/week
1½ year to 2½ year	2/month	2/month	4/year	4/year	4/year	4/year	4/year	2/month	Continuous	1/week
2½ year to 6½ year	6/year	6/year	2/year	2/year	2/year	2/year	2/year	6/year	Continuous	2/month
Subsequent years	2/year	2/year	1/year	1/year	1/year	1/year	1/year	2/year	Continuous	1/month

Table 18.9. Monitoring for Swiss dams (Swiss National Committee in ICOLD 1989).

In the normal case it is usual to carry out measurements at the following intervals:

Visual inspection weekly or every two weeks:
Exceptions to this rule are admissible in all cases where access is difficult, for example in winter, or when the reservoir is not full

Important measurements monthly (sometimes twice monthly when the reservoir is near its maximum level):
Plumb-lines measurements
Concrete temperature (when needed for computing compensated displacements)
Settlement of embankment dams; rockmeter measurements (extensometers, invar wires)
Uplift pressure, pore pressure; flow measurements (seepage, drains, springs)
Movement of certain cracks

Twice yearly (under the same seasonal conditions and at the same reservoir levels):
Joint opening; clinometer measurements

Annually:
Detailed inspection by a civil engineering expert (if possible always the same one)
Trial operation of bottom outlet gates with the reservoir full (small opening with water flow, full opening without flow)
Trial operation of spillway gates (if they have not been operated during the past year)

Every 5 years:
Geodetical measurements (but with the possibility of carrying out these measurements without delay in case of unusual events)
Five yearly evaluation report by an independent and recognized dam construction specialist – Inspection of condition and behaviour of the dam, its foundations and surroundings; analysis of long-term structural behaviour; inspection of monitoring and surveillance system; if necessary a check on dam safety by new computations, or a check on new flood criteria

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Subject index

The three authors of this book each have more than 25 years experience in geotechnical aspects of site selection, design, construction and surveillance of dams in Australia, Asia and the Pacific. This has been tempered in recent years by applied research and academic activities. The result is a text with a blend of theory and practice, which deals with the engineering of all types of embankment dams. The book is well illustrated and is suitable for practitioners in civil and geotechnical engineering, and in engineering geology. It is also suitable as a text for graduate and undergraduate students.

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