ROCKFILL DAMS
Design and Construction

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Rockfill Dams
Design and Construction
by Björn Kjærnsli, Tore Valstad and Kaare Höeg
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HYDROPOWER DEVELOPMENT

VOLUME NO 10

ROCKFILL DAMS

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Norwegian Geotechnical Institute

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An Introduction to the Series

Hydropower planning covers a wide range of topics. Knowledge of technology, economics and the environment is needed. Another requirement is for planning skills which largely depend on professional expertise in the field. The transfer of experience to younger professionals has traditionally come about through close and continuous working relationships between older and younger colleagues.

Not all of this knowledge has been properly documented and there are few, if any, recent textbooks dealing with all the topics involved. Most recent publications in the field have been based on case studies. This situation has been the impetus for the present work. Compiling this comprehensive and systematic documentation on the planning, design, construction and operation of hydropower plants has been a challenging and time-consuming task.

In many industrialized countries professional expertise on hydropower is disappearing as the countries' hydropower resources are becoming fully exploited. However, Norway and many developing countries still have numerous unexploited hydropower resources. Since hydropower is a clean and renewable source of energy when properly planned and managed, these resources will be of increasing importance in the future. Safeguarding and recording our present knowledge and experience in hydropower engineering are therefore matters of global significance.

The Norwegian Contribution to Hydropower

Although hydropower technology is global in scope, the series reflects primarily Norwegian experience. This is due to Norway's extensive exploitation of hydropower resources, encouraged by the local topography, geology and climate.

Most of the country is mountainous, and in recent geological times, mighty glaciers formed numerous lakes at high altitudes. This means that even small dams can create sizeable storage volumes which can be further increased by underwater "lake-tapping" – a technique pioneered by Norwegian engineers.

The country's geology provides a bonus since its mountains mainly consist of solid rock, which makes the construction of underground power stations relatively inexpensive.

Norway's climate should also be considered. The country's annual average precipitation of 1.400 mm has very uneven geographical distribution. During the winter, when the demand for electric power is greatest, much of the precipitation is retained on the ground as snow.

As the cheapest and most readily available waterfalls have been exploited, necessity has become the mother of invention forcing design engineers to find new solutions and adopt improved technologies. Such schemes have not only broken new ground, but are also economically viable. Norway's particular natural characteristics and the pressing demand for economical design have given rise to methods and solutions which represent major advances in hydropower technology and which are widely applicable to hydropower schemes in other countries.

The embodiment of Norwegian hydropower experience in this series not only meets the needs of Norwegian engineers but it will also be of great value to engineers engaged in the planning, construction and operation of hydropower plants elsewhere in the world.

About the Volumes

All volumes in the series are presented on the back cover of each volume. Each volume deals with a separate topic. The theoretical basis for each topic is set out and followed by a description of how it is applied in each discipline in practice. Each volume is thus a distillation of current thinking and practice within the particular topic or field.

The series is planned so that its volumes can be combined to meet the needs of different groups of readers. It should be noted that most of the content of each volume is original and has not been published before in existing textbooks. This means that when planning a complete course in hydropower engineering, this series will provide a valuable supplement to conventional textbooks.

Acknowledgements

The authors of the respective volumes are all leading professionals within their fields. The Editing Committee wants to extend to each of them its most sincere thanks for outstanding and unselfish efforts in realizing this project.

The Committee also wants to express its gratitude to all financial supporters whose contribution made the realization of the project possible.

Trondheim 1992
Editing Committee

PREFACE

The Norwegian Geotechnical Institute (NGI) has actively been engaged in the planning, design, construction control and monitoring of some 110 large rockfill dams up to 145 m high. The authors are all members of the NGI staff, and the senior author has headed up these activities at NGI, including the accompanying research and development relating to embankment dams, since 1953.

The manuscript for this book is based on an NGI design manual. The objective is to present practical design and construction recommendations. The material is mainly based on Norwegian practice and conditions, but also on extensive international experience.

Chapter 1 gives a summary of experience, recommendations and views with reference to further discussion in the subsequent seven chapters. Textbooks should be consulted for details on theoretical soil and rock mechanics and on the use of numerical methods for analysis of deformations, stresses and fluid flow in porous media. Emphasis is placed on the exercise of judgment and the importance of personal interaction between designers and field forces.

The authors would like to thank all their colleagues at NGI who have contributed to the development in the laboratory and in the field, in theory and in practice.

NGI, 1992

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Moraine or glacial till, which Norway possesses in abundance, has been the predominant material used in the construction of the impervious element. However, in the past decade asphaltic concrete cores have come increasingly into use. They are less susceptible to adverse weather, which makes construction work easier, and potential scars in the landscape from borrow pits are avoided. In Norway today, there is a continuous process of research and development in the design and construction of rockfill dams.

1.2 Design of Impervious Element

Various terms used in connection with embankment dams are explained in Figure 1.3.

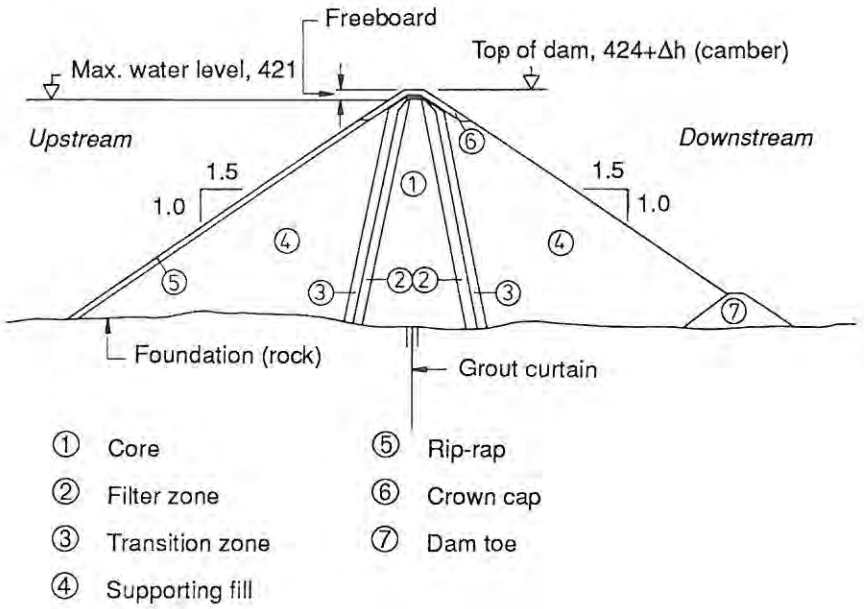


Fig. 1.3
Explanation of terms relating to embankment dams. Cross-section of a typical rockfill dam with a central core of moraine.

Impervious elements may be designed by use of earth, clay, concrete, asphaltic concrete, bitumen or even steel, wood and more recently synthetic membranes. The impervious element may be embedded within the fill or placed on the surface of the upstream slope of the embankment. Depending on the type and placement of the impervious element, the dam may for instance be characterized as a rockfill dam with central moraine core, or a rockfill dam with concrete facing. Various types of embankment dams described by cross-section are shown in Figure 1.4.

As shown in Table 1.1 most Norwegian embankment dams are rockfill dams with impervious core of moraine. Moraine or glacial till is a scoured material, deposited beneath the ice during the last glaciation, it is a broadly graded mixture of boulders, stones, gravel, sand, silt and clay.

A typical rockfill dam is shown in Figure 1.3. The cross-section comprises of: 1) impervious core of moraine, 2) filter zones of sandy gravel, 3) transition zones of fine blasted rock, and 4) supporting shells of blasted rock.

Moraine cores are so widely used as the impervious element of embankment dams in Norway because deposits of loose material in mountain areas

where most dams are situated, usually consist of either glacial tills or fluvial deposits. The fluvial deposits are sand or gravel which may be suitable filter materials, but are too permeable for waterproofing. Moraine deposits on the other hand have the fines content required for materials of sufficiently low permeability.

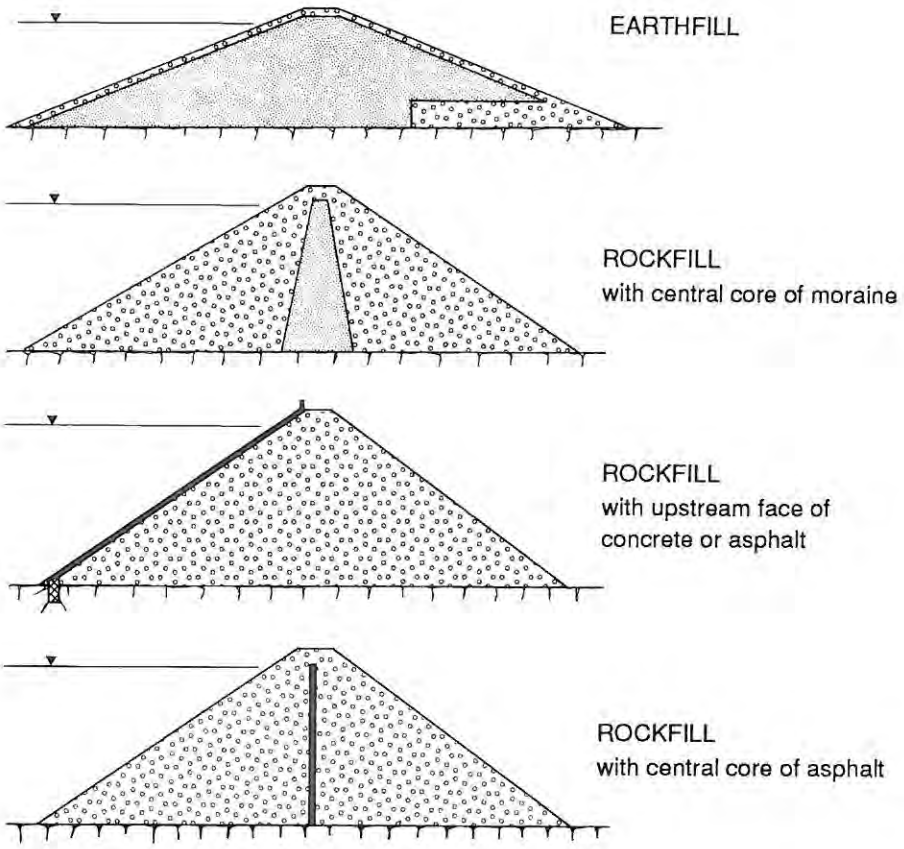


Fig. 1.4
Typical cross-section design of embankment dams

In places where suitable moraines do not exist within an economic hauling distance from the dam site, other impervious materials have to be employed. Examples of dams with various alternatives are shown in Figure 1.5. Marine clay, for instance, was used in the frontal blanket of the Mänika Dam as shown on top of the figure.

Marine clay is found in areas raised above the present sea level after the last glaciation. In Norway, the land has risen most, about 220 m, in the south-east. In other regions the rise varies from 25 m along the western coast to about 100 m in mid-Norway and 25 to 75 m in the North.

In some cases it may prove economical to crush soft rock to sufficient fineness for use as impervious material. Crushed phyllite has been utilized for impervious cores in two dams in Western Norway.

Several dams have impervious facing on the upstream slope ("frontal decks"). Materials used in the facings are asphaltic concrete (Venemo), wooden planks (Aursjø) and reinforced concrete (Grøndalsvatn). During the 1980s several rockfill dams were built with a central core of asphaltic concrete as decribed in Section 7.7.

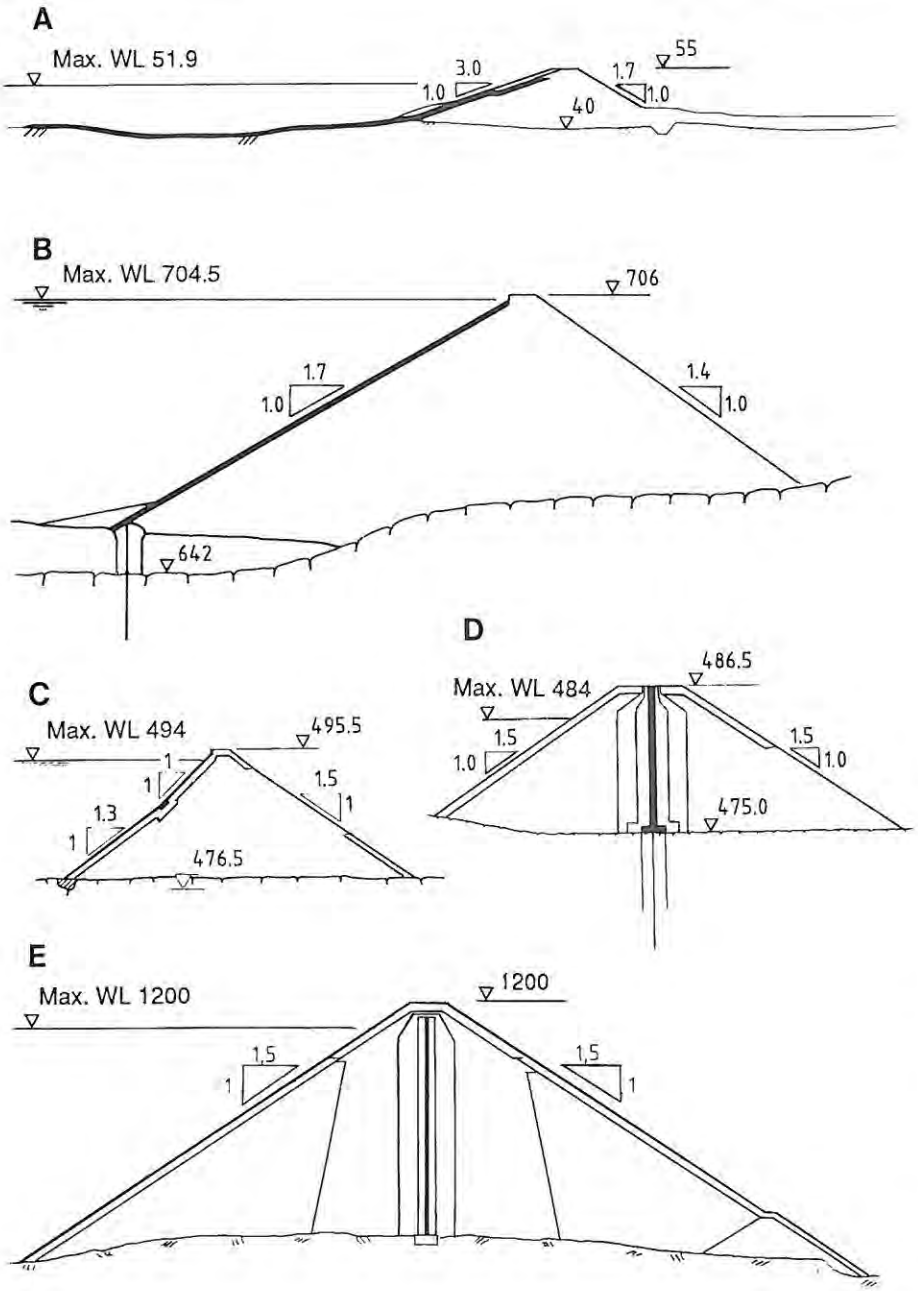


Fig. 1.5
Examples of Norwegian embankment dams with impervious materials other than moraine:

A) Lille Mänika Dam, frontal blanket of marine clay. Completed 1964.

B) Venemo Dam, frontal deck of 15 cm asphaltic concrete. Completed 1963.

C) Grøndalsvatn Dam, frontal deck of cement concrete. Completed 1971.

D) Gråsjø Dam, test section with stone-bitumen core. Completed 1969.

E) Styggevatn Dam, central core of asphaltic concrete. Completed 1990.

1.3 Control of Foundation Seepage

The impervious element, be it a moraine core, a concrete deck or an asphaltic concrete core wall, must keep seepage through the dam to a minimum. To prevent excessive seepage through the existing ground, the impervious element must be extended down into the foundation underneath and possibly to the sides of the dam. This may be achieved by different types of cut-off walls or grouting of overburden and rock foundation.

The extent of grouting needed is assessed by means of borehole pumping tests. The grouting of a rock foundation is usually carried out from boreholes aligned in three rows along the centre line of the core foundation. The holes in the central row are drilled deeper than the holes in the other rows (Section 6.2).

In many countries it is common practice to build a gallery under the impervious core for the purpose of inspection and drainage control and to facilitate grouting, if required, after the embankment is completed. None of the Norwegian large embankment dams have such a gallery. The cost-benefit effect has not been found favourable, and the gallery has not been deemed necessary or desirable from a safety point of view (see Section 6.3).

Although needing grouting, rock presents the simplest type of foundation works. If the rock is covered by overburden, it may be feasible to excavate a foundation trench down to the rock surface in order to secure a proper base for the impervious core. The supporting shells, however, can often be safely founded on existing overburden. This was done at Arstaddalen Dam (Figure 1.6a).

If the overburden is of great thickness, but consists of impervious moraine, the entire dam may be built upon the moraine. Examples of such foundations are presented in Figure 1.6. In the cases of Songa II and Manndøla, grouting of the substrata was carried out. Other measures are also usually required such as proper drains and filters as shown at Sønstevatn to control seepage forces and exit gradients (Figure 1.6e).

Compared with dams on rock, dams founded on overburden will generally require flatter embankment slope angles. A special case of a foundation on loose deposits is represented by Mänika Dam (Figure 1.5a). The overburden consists of sand and silt to great depths. Since the quantity of seepage was unlikely to be of any major significance from an economic point of view, the dam was built upon the deposits without any foundation seepage cut-off. To limit leakage and prevent downstream erosion, the frontal blanket of marine, sensitive clay was extended upstream into the reservoir.

During impounding, the seepage, running at a rate of 140 l/s, caused spring-water boils to create numerous small heaps of eroded material downstream of the Mänika Dam. To stop the erosion from causing increasing leakage, the ground downstream was covered with a layer of sandy gravel to provide filtering ballast. Seepage rates have since 1965 remained practically unchanged at about 80 l/s, which is satisfactory for this particular project.

In contrast to the situation at Mänika Dam, it will normally be economical to aim for minimal seepage. In most Norwegian embankment dams seepage amounts to a few litres per second at the most, which means that they are practically watertight.

1.4 Dam Deformations and Displacements

During construction embankment materials previously put in place and compacted will be subject to increasing loads from the weight of the subsequently placed layers. These materials are being further compressed as the embankment grows. Most of the settlements therefore, take place during construction, but even after the dam is completed, settlements will continue. Settlements at the top of a number of dams are shown in Figure 1.7, which illustrates that settlements continue for many years.

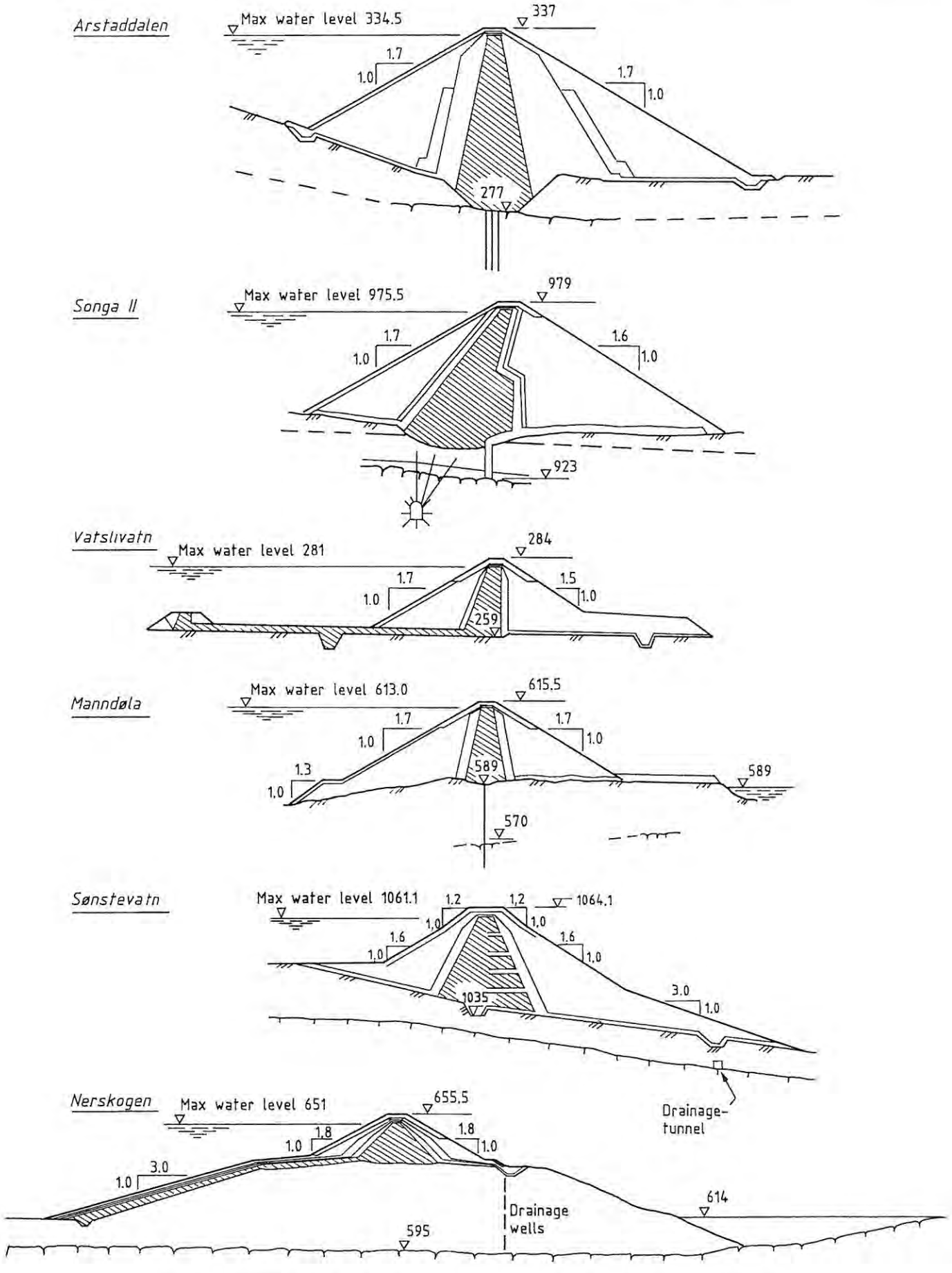
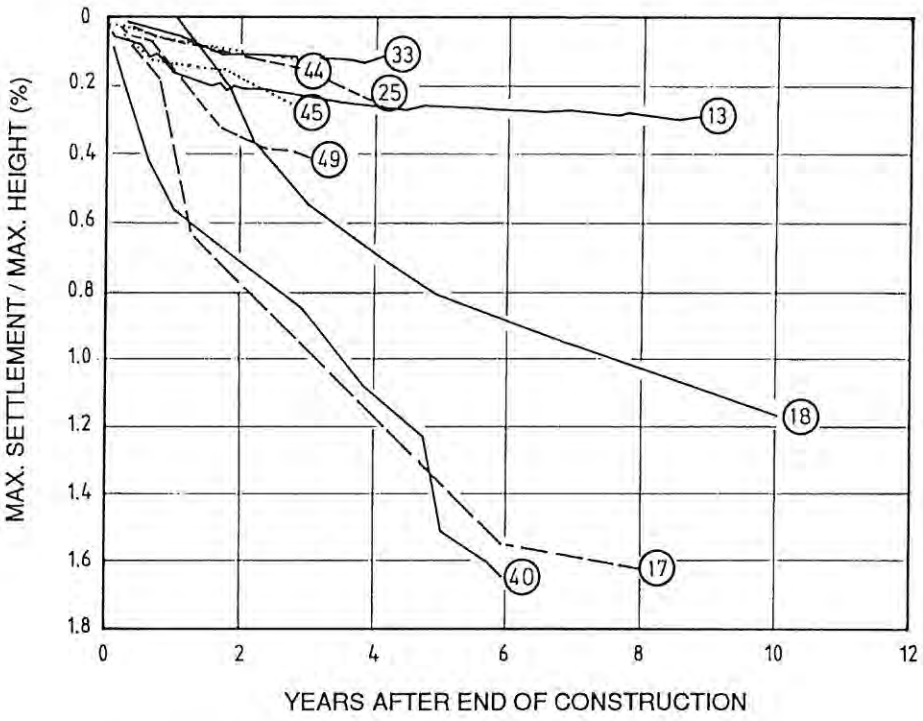


Fig. 1.6 Examples of embankment dams founded on left-in-place overburden

Some older dams have developed settlements significantly larger than others and more than expected, reducing the freeboard beneath statutory levels. Consequently the top of these dams has had to be rebuilt to restore proper freeboard with respect to the highest regulated water level.



13	Hyttejuvet	25	Roskreppfjorden	44	Nesjø
17	Akersvatn	33	Digeå	45	Finkoihøgda
18	Kalvatn	40	Muravatn	49	Viddalsvatn

Fig. 1.7
Crown settlement as a percentage of dam height measured on some rockfill dams with moraine cores resting on bedrock

Measurements show that vertical and horizontal displacements change with time and with varying water levels in the reservoir. As an example, Figure 1.8 shows measured displacements in the 129 m high Svartevann Dam (see front cover photo).

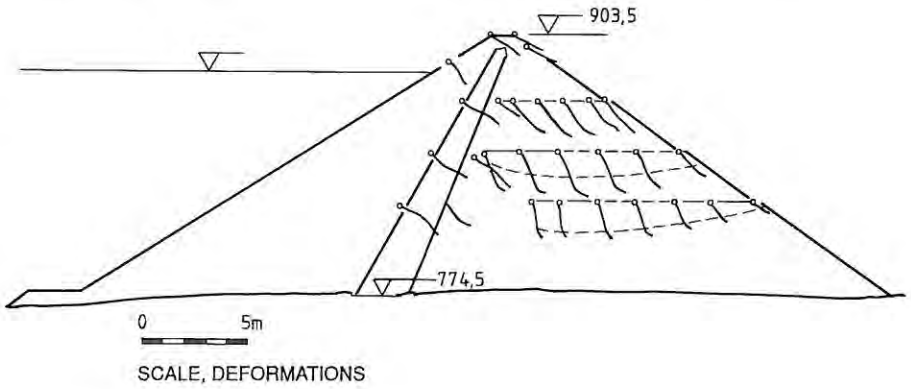


Fig. 1.8
Deformations measured in the largest cross-section of Svartevann Dam, as per September 1980. The dashed lines indicate level of displacements developed during construction and simultaneous impounding.

Embankment deformations and displacements are discussed in Chapter 5, and instrumentation and measuring methods are presented in Chapter 8. The important effects of placement and compaction on reducing the magnitudes of displacements and decreasing the potential for cracking and hydraulic fracturing, are described in Chapters 5 and 7.

1.5 Unexpected Leaks

A few dams have developed undesirable and unexpectedly large leaks during the first filling of the reservoir, e.g. Hyttejuvet, Viddalsvatn and Mysevatn dams.

In Viddalsvatn Dam leakage rates rose and fell with variations which were disproportionate to changes in water level, as shown in Figure 1.9. The leakage water became murky with dispersed particles during the short periods when leakage increased. Leakage most probably developed through hydraulic fractures in the core caused by unfavourable redistribution of stresses and strains within the embankment. The concentrated leaks caused some erosion as evidenced by piping holes through the core and swallow holes at the top upstream surface (Figure 1.10).

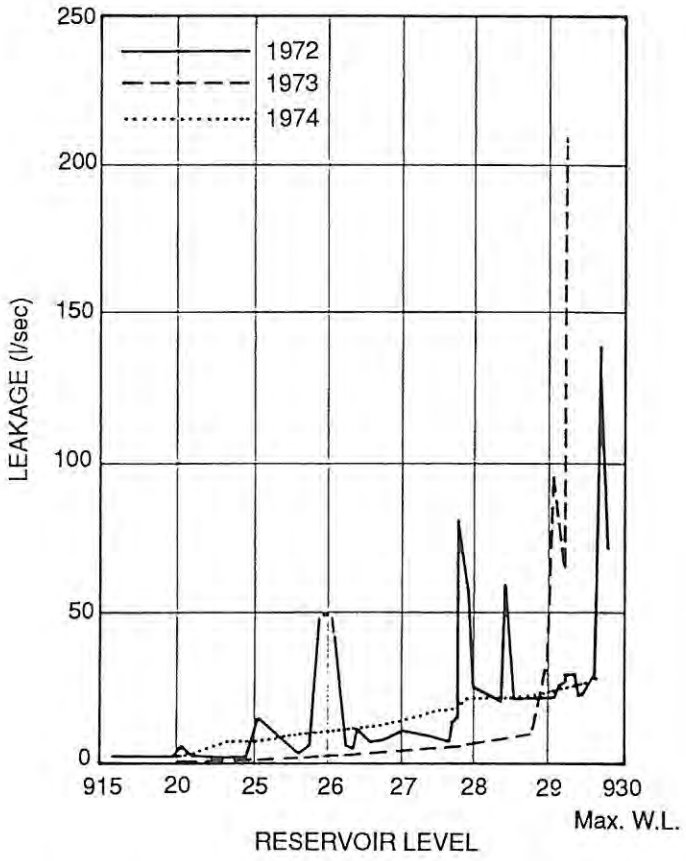


Fig. 1.9
Viddalsvatn Dam.
Leakage rates and water levels in the reservoir.
(After Vestad, 1976)

Similarly, in the Hyttejuvet Dam swallow holes and sink holes may have been due to internal erosion. The subsidence of about 30 m^3 of moraine took place within a vertical cylinder about 2.5 m in diameter. Swallow holes also appeared in Mysevatn Dam, and subsidence occurred at the upstream surface near the top of the dam.

But how do these dams differ from all the other rockfill dams which have not exhibited significant leaks or this type of subsidence? It shall be noted that Viddalsvatn and Hyttejuvet have cross valley profiles which are remarkably similar. The left valley abutment is very steep, but relatively even. The right abutment is equally steep near the valley bottom, but flattens out with gentler, more variable slopes. The steep and uneven slopes

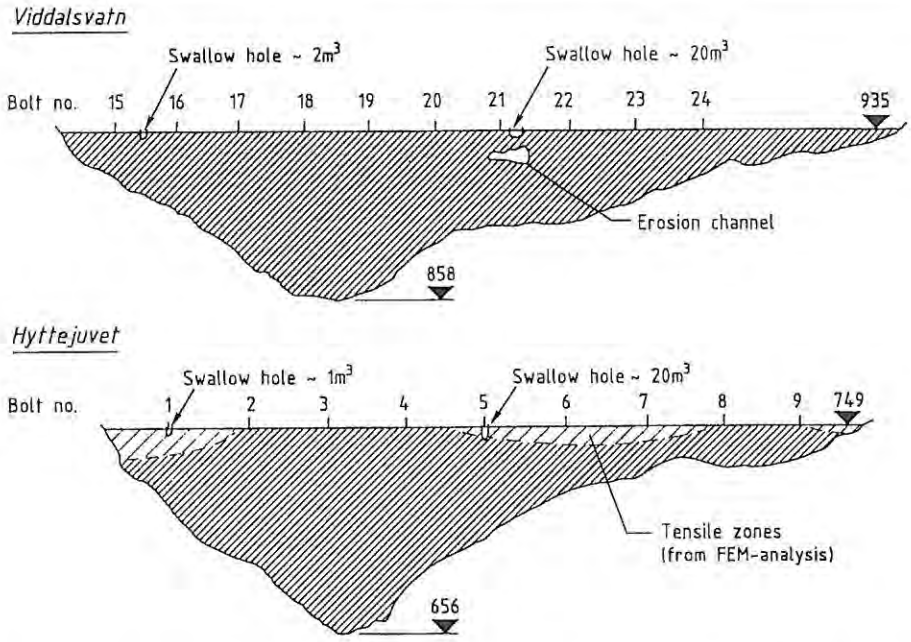


Fig. 1.10
The cross valley profiles of Viddalsvatn Dam and Hyttejuvet Dam are very similar. The location of sink holes coincide with extension zones computed by finite element analyses.

with ridges may have aggravated deformations in the longitudinal, cross-valley plane through the dams. The swallow holes are similarly located in both dams with the largest subsidence taking place above the flatter slope on the right hand side of the valley (Wood et al., 1976).

Both dams were built very rapidly. The embankments were raised to full height in the course of two construction seasons at a rate of 40 to 50 m per season (a construction season in Norway amounts to about 5 months). Furthermore, in both cases impounding did not begin until the dams were completed. The water level then rose very rapidly, by as much as 8 m per week at Hyttejuvet, where the average rise during the first filling of the reservoir was 4 m per week.

Experience has shown that the rate of reservoir filling may critically affect the development of cracks in the core. A slow and gradual filling of the reservoir is less taxing to the dam than a rapid filling, as the dam structure has time to adjust to the change in loading. Controlling the rate of filling will often depend on the ability to release water from the reservoir during the first filling. The performance and capacity of the outlet structures are therefore essential for the real safety of the dam.

Mysevatn Dam (Figure 1.11), besides having a narrow top width and a steep embankment slope on the downstream side, is sited on topographically unfavourable rock ridges. The topography also imposed a sharp, almost 90° downstream convex bend in the dam axis. The radial displacements of the core due to water pressure will therefore have introduced longitudinal strains which tend to open transverse cracks in the curved part of the core. The unfavourable topographical circumstances at Mysevatn together with the slenderness of the dam have increased deformations and differential strains in the dam. This may locally have reduced compressive stresses so that cracking may have occurred in the core. (The top of the dam was widened and the downstream slope angle reduced in 1978.)

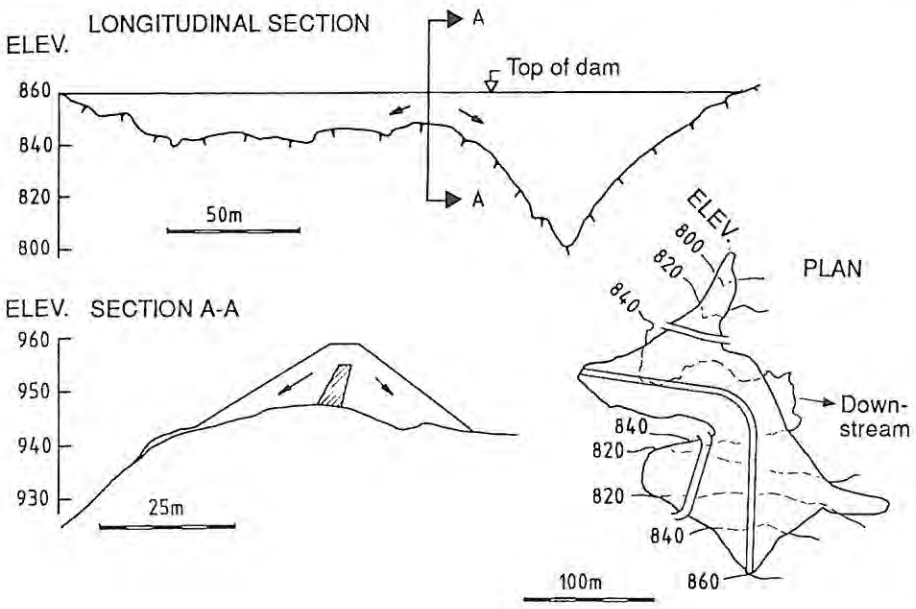


Fig. 1.11
The Mysevatn dam is situated in very unfavourable terrain

In order to reduce the leakages in all three of the dams described above, grouting of the core was carried out through grout holes from the top. The grouting mix consisted of a slurry of cement, bentonite, sand and water. This mixture cannot penetrate the morainic material, but is squeezed along existing cracks in the core. Cracks are filled under pressure, and this will displace and compress the adjacent core material, thus consolidating the core. As a result of the remedial work, the leakages have been substantially reduced and all three dams have since performed satisfactorily.

It is important for further progress in embankment dam design and construction that unsatisfactory performance is recorded and reported to the profession. The main purpose of the above discussion on the problems affecting the dams at Viddalsvatn, Hyttejuvet and Mysevatn is to make it clear that embankment dams do have their weaknesses which must be carefully considered. Unsatisfactory performance is rarely attributable to a single cause, but stems most often from a combination of unfavourable factors collectively producing effects which are difficult to quantify or assess at the planning stage. To avoid such unfavourable combinations it is important to recognize potentially suspect materials and adverse conditions at the site. Precautionary measures should be included in the design and implemented as necessary during construction (see Section 6.10). Filter design and adequate drainage capacity are two essential aspects.

1.6 Filter Protection and Drainage Capacity

All dams must be designed so that unexpected leakage cannot lead to failure. This is an essential design aspect. Water percolating through granular materials will pull along particles of the material in its path; the higher the flow velocity, the greater is the size of particles that may be carried away. To prevent progressive erosion of the core, it must be protected by a filter capable of stopping transported particles without stemming the water flow. To meet these requirements the filter material, which usually consists of sand and gravel, must be sufficiently fine-grained to retain eroded particles

of a certain size, but at the same time coarse enough to discharge seepage water freely. The filter material must also be "cohesionless" so that the filter does not develop open cracks within itself, but will collapse and close the gaps should cracks tend to develop (Section 2.3).

As a filter is itself susceptible to erosion by the flow of the water, it must in turn be protected by a coarser filter (or filters), usually called a transition zone, providing a link between the finer gravel of the filter and the coarse rockfill of the supporting shoulder. Figure 1.3 shows the cross-section of a typical rockfill dam. It is provided with filter and transition zones on both sides of the core. The purpose of downstream filters is, as mentioned above, to filter seepage water. Similarly, the purpose of upstream filters is to prevent the washing out of fines by waves and sudden drawdowns of the reservoir level. Furthermore, should concentrated leaks develop in the core, the upstream filter should provide eroded material which would curtail and eventually stop the leakage by clogging the flow paths, thus enhancing the dam's capability of "self healing".

Appropriate filter and transition zones are the most effective safeguards against dam failures due to excessive leakage. To prevent unravelling of the embankment by exiting water from major leaks, the dam toe and the surface of the downstream slope should consist of large selected stones. These should be placed upon a non-erodible base and remain stable even under intense throughflow of water (Section 6.6). Thus, adequate drainage capacity is provided.

Figure 1.12 gives an example of grain-size distribution curves for materials used in various zones of rockfill dams. The diagram shows that broadly graded materials are used so that coarseness increases step by step from the impervious material of the core to the blasted rock material of the outer shells.

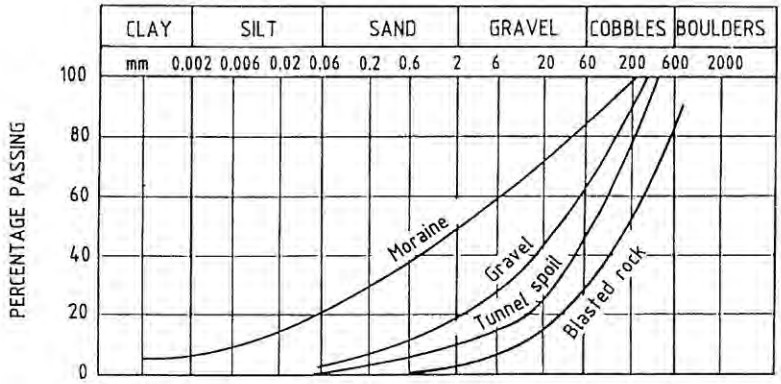


Fig. 1.12
Typical grain-size distribution curves of core, filter, transition and supporting fill materials in Norwegian rockfill dams.

1.7 Wave Damage to Upstream Slope Protection

Damage by waves has occurred, especially during autumn gales while water is at its highest level. Damage of the slope protection layer has, in the worst cases, triggered small slides in the upper parts of the upstream slope.

Stones in the outer layer of the slope must be of sufficient weight to withstand wave action. They must also be resistant to frost action and weather-

ing, and openings between stones must not allow adjacent materials to be washed away. Large stones must therefore be interspersed with smaller stones to reduce the size of pores. In cases where damage did occur, these requirements had been neglected, and the ensuing maintenance costs proved to be high. Fairly detailed analyses and design considerations related to wave action are therefore presented in Section 6.7.

1.8 Dam Failures Do Occur

In addition to the 172 embankment dams higher than 15 m in Norway there are several thousand smaller dams with heights of a few metres. None of the high dams have failed, and the defects have been minor. This excellent record is undoubtedly related to the fact that most Norwegian embankment dams are rockfill dams founded on rock.

Several small and very small dams have failed, however. According to historic records, failures occurred at an average rate of one per year over the period 1920 to 1970, costing in all six human lives. Most of these failures were embankment dams of earthfill for which dam engineers had not been engaged, and overtopping was the most common cause.

An international survey shows that the mechanisms of registered dam failures are usually complex, and little is known about the course of events leading to failure. In some instances the entire dam was washed away following the breach. When, at the same time, proper records on design and construction data are missing, it has been impossible to trace the causes. In many cases there was no documentation of the design, making statistical classification impossible. Nevertheless it is known that many such dam failures were caused by overtopping. The next most important cause was that leaks caused erosion and subsequent slides and wash-outs through the dam itself, or through loose ground underneath.

In 1976 the Teton Dam in the USA collapsed during first filling of the reservoir. The 90 m high zoned earthfill dam was constructed from fine grained materials of clay, silt, sand and gravel and a very small amount of stones. The series of photos in Figure 1.13 shows that erosion started near the right abutment with water emerging in the lower part of the downstream slope. Piping holes opened up rapidly and widened to form a large tunnel through the embankment. Within hours, the tunnel subsequently collapsed, forming a deep breach in the embankment through which the collapsing masses were swept away. In the book "Dam failures" (1987), several experts discuss the failure of the Teton Dam and the Baldwin Hills Reservoir, as well as the Malpasset Dam and the catastrophic slide into the Vajont Reservoir.

Sowers (1977) has studied the failure of numerous dams, both large and small. His findings are reproduced in Table 1.2 which presents a simple statistical treatment by dividing the types and causes of failures into three principal groups. Each of these groups is further sub-divided to identify more specifically the development and causes of the failure. Precautionary measures which might have prevented the failure are listed in an adjacent



Fig. 1.13 Erosion caused complete failure of the Teton dam, USA in 1976. (After Independent Panel, 1976)

Table 1.2
Statistical survey of development and causes of failure of embankment dams. Freely after Sowers (1977).

Type of failure	Characteristics of failure and causes	Preventive or corrective measures	Statistical distribution	
			Large dams	Small dams
External erosion	Flow over embankment (overtopping) as a result of insufficient freeboard, inadequate or clogged spillway. Wave erosion as a result of insufficient slope protection. Erosion of downstream slope as a result of heavy rain.	Increase freeboard and camber (settlement reserve), increase spillway capacity and prevent clogging of spillway. Increase thickness and improve quality of slope protection. Protection of downstream slope. (Not applicable to rockfill dams.)	20%	50%
Internal erosion	Erosion in downstream side of dam foundation progressing backward towards the upstream side. The erosion develops into an open conduit or "pipe" that often leads to a washout of a section of a dam.	Reduce the possibilities for cracking, excavate the foundation to a favourable surface shape and give special attention to culverts and other concrete structures passing through the dam. Protect all possible seepage sources and paths by filters and collect water into stable drainage conduits.	50%	30%
Slope slippage	Slippage in upstream and/or downstream direction as a result of insufficient bearing capacity in the foundation. Slippage in upstream direction as a result of rapid drawdown of the reservoir. Slippage in downstream direction as a result of high pore pressures in the embankment or in the foundation as a result of water seepage.	Flattening of the slopes, employ stabilizing berms or remove weak materials. Provide drainage wells or drainage trenches to reduce pore pressures in the foundation. Flattening of upstream slope and/or provide drainage layers in upstream shoulder. Flattening of downstream slope and/or drainage of downstream shoulder. Provide seepage control measures in the foundation.	30%	20%

column. The last two columns indicate the percentage of failures due to each cause for large and small dams, respectively.

As regards Norwegian large dams, the defects which have occurred mainly stem from external and internal erosion. No dams completed since 1975 have shown any sign of deterioration.

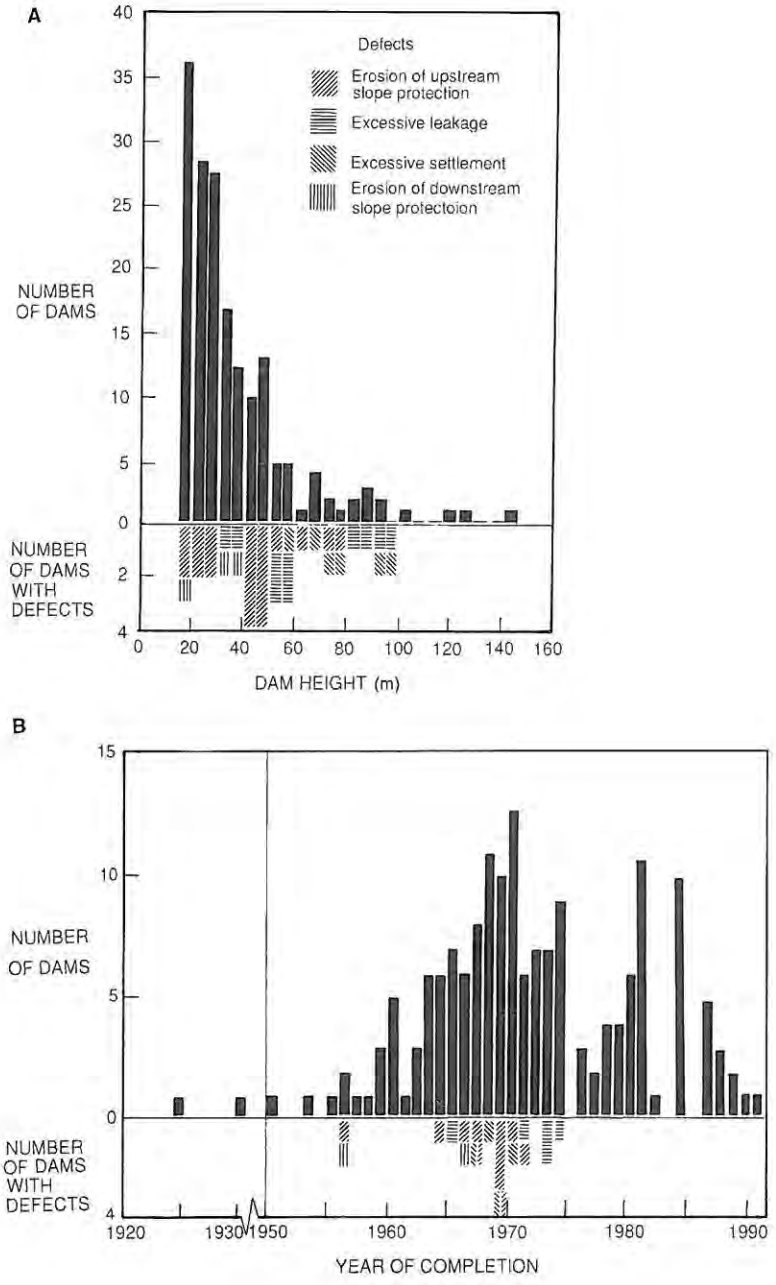


Fig. 1.14
Number of "defect incidents" distributed according to height of dam (A) and year of completion (B) for Norwegian embankment dams higher than 15 m

As described in Section 1.5, it is very important to be able to control the water level in the reservoir, which depends on the ability to release water. A controlled rate of filling and lowering, if required, is particularly important during first impoundment. The performance and release capacity of the outlet structures are essential to the safety of any dam. It is not advisable, as is often done, to plug diversion tunnels so they cannot be operated by gates or valves after dam construction is completed, unless adquate release capacity is ensured through other structures.

2.
DAM SITE, MATERIAL SOURCES AND REQUIREMENTS

In some cases it may suffice to base the preliminary design on available information and on visual inspection of the site and potential material sources. In other cases more detailed field investigations may be needed even for the preliminary planning of the dam. The scope and methods used for field investigation for a final project design are outlined in the following.

Topographical and geological mapping of the site is required to determine the most favourable layout for the dam construction. The quality of the foundation rock and the thickness and properties of overburden soil have to be investigated.

Potential sources of fill materials should be explored. There must be sufficient quantities and suitable qualities for the various zones of the dam. The suitability of earth material deposits is determined by grain-size distribution and mineralogical composition of the material.

The properties of rockfill materials can be judged on the basis of parent rock quality and fragmentation which may, however, be heavily influenced by the blasting method. The grain-size distribution of the tunnel spoil will also depend on the cross-sectional size of the tunnel.

When the rock foundation surface is exposed, or lightly covered by overburden, the quality of the rock foundation may be investigated by geological mapping. For major projects the quality should be confirmed by drilling holes for core sampling and water pressure tests. Such borings can often be postponed until the site is accessible by road and are then conveniently carried out as a first step in the grouting procedures.

2.1 Permeability Testing of Rock Foundations

The rock foundation permeability is determined in accordance with international, standard methods. Leakage from a borehole, diameter 50 to 80 mm, is measured by pumping water under given pressure through the wall of a given length of the borehole. The test length may be established between a packer and the bottom of the hole or between two packers. To test separate sections, the packers are re-positioned stepwise in the hole. Each separate section of the hole is exposed to constant water pressure which is measured

by a manometer attached to the water hose at the entrance of the hole. The loss of water over a 5 minutes period is measured by a flowmeter. Repeated measurements are taken until two consecutive readings give identical results.

The test results are given in Lugeon-units which equal 1 litre per minute per metre borehole at 10 bars overpressure. When different overpressures are applied, the water loss is considered to be directly proportional to the overpressure:

$$1\ L\ (Lugeon) = \frac{Q}{t\,\ell}\ \frac{p}{p_0}$$

where Q : water loss in litres

t : test duration in minutes

ℓ : length of test section in metres

p_0 : reference pressure equal to 10 bars (1 MPa)

p : applied pressure at top of borehole (bar).

Assuming water loss is evenly distributed through a mesh of similar cracks in the adjacent rock mass, one may convert the Lugeon-value to a coefficient of permeability expressed as:

$$1\ L \approx 1.5 \cdot 10^{-7}\ m/s$$

Dam foundations are generally considered to be sufficiently impervious if the Lugeon values do not exceed 1-4 L (Section 7. 2). The foundation may then be slightly more permeable than a moraine core where permeability usually would be in the order of 10^{-7}-10^{-9} m/s.

Testing the same section at different overpressures may show that water loss will increase disproportionately when high enough overpressures are applied, as the pressure exerted widens existing cracks or creates new ones. Disproportionate leakage can also arise as increasing leakage under higher pressure washes out fines from existing crack fillings. Figure 2. 1 shows the typical variations in pumping test results depending on applied pumping pressure and its effects on the ground being tested.

2.2 Sources of Fill Materials

A rockfill dam with a moraine core is the least expensive type of dam provided suitable moraine and gravels are available sufficiently close to the dam site. What constitutes sufficiently close will depend on local circumstances. If, for instance, moraine can be located within a distance of about 20 km along an existing road, or a road is to be built anyway, a moraine core may be economical. If the borrow pit requires the construction of a new road access, economic hauling distances become very much dependent on the size of the dam to be constructed.

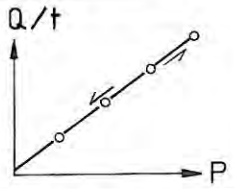
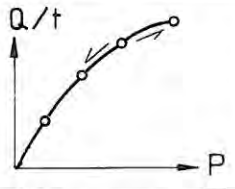
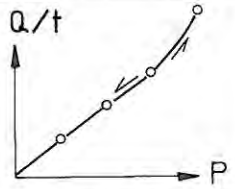
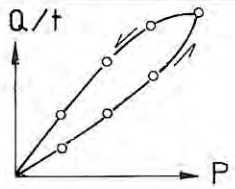
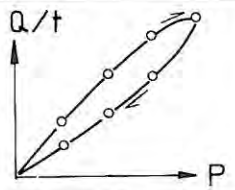
LEAKAGE (Q/t) AT VARIABLE PRESSURE (P)	CHANGE IN CONDUCTIVITY IN JOINT SYSTEMS	PHYSICAL EXPLANATION OF THE OBSERVED CHANGES
	None	Laminar flow
		Turbulent flow at high pressures
	Temporary	Elastic deformation of joints at high pressures
	Permanent	Outwash of fines or plastic deformation of joints
		Test performed before static conditions established or blocking of joints

Fig. 2.1
Suggested explanations for alterations in conductivity at different pressure levels

An initial survey of prospective borrow pits is required in order to set up a program for further field investigations. Prior to the field trips it is useful to gather any available information from existing aerial photos of the area and to acquire knowledge about existing or planned roads in the vicinity of the dam.

In the case of projects of any size the quaternary geology should be mapped. Types and origins of deposits may indicate probable material properties such as grain-size distribution, fines and water content and thus provide basic information for the planning of further investigations. For instance, bottom and front moraines are often of a more even quality and require less investigation than an ablation moraine. Where formations of loose sediments within small areas vary from glacial to fluvial deposits, variations can be extremely complex and difficult to classify. In such cases unpleasant surprises can be expected, even after extensive investigations have been carried out.

As regards rock quarries, visual inspection will generally be sufficient to locate moderately fissured rock formations of a suitable type for rockfills.

2.3 Core and Filter Materials

Materials for core and filter have to meet certain requirements for grain-size distribution. A wide variety of impervious materials have been utilized in dams around the world and absolute criteria or rules of general validity for such materials cannot be established. However, for moraines commonly used in Norway, some general rules can be put forward as examples of specification requirements.

The content of fines must be sufficient to secure a reasonably low permeability, and the content of stones must be limited so as to prevent segregation of stone surplus in the core. The maximum size of the stones should be restricted, normally to less than two thirds of the actual placement layer thickness. The requirements for the content and size of stones can be met by screening. When the stone content is low, oversized stones can simply be picked out during excavation and/or placing of the material.

The content of fines, however, is given by the natural state of the materials. Any change in the degree of fines by artificial means will not usually prove economical. A common specification for required fineness of moraine is that at least 15% of the material of grain-size 19 mm and smaller shall pass through a standard 0. 075 mm gauge meshed sieve.

Only a lower limit and no upper limit is therefore set for the content of fines. Attention has, however, to be paid to construction difficulties arising from an excessively high content of fines. Such material is highly susceptible to climatic wetting and can become unusable or at least very difficult to handle with an increased water content. The combined effects of high fines content and high water content may result in a core with unfavourable properties. A high content of fines in the core will correspondingly require fine-grained filter and transition zones as explained below.

Dispersive soils are materials that under certain conditions deflocculate and are rapidly eroded and carried away by water flow. The use of such soils has resulted in failures of homogeneous embankment dams. Even minor hydraulic fractures may lead to failure by progressive erosion (Sherard, 1986). However, protected by properly designed filters, dispersive soils may be used as core materials. Detailed recommendations are given in ICOLD (1990), which also describes laboratory procedures for testing soils with dispersive properties. So far, dispersive soils have not been of concern for construction of embankment dams in Norway.

Filter materials must fulfil certain grain-size distribution criteria. The grains must be able to resist compaction during construction without breaking to an extent which would significantly change the initial grain-size distribution. In addition, the grains must not weather and crumble in the life-span of the dam.

In the international literature various filter criteria are presented, all specifying acceptable grain-size distribution limits for filter materials. A common starting point for most of those criteria is the grain-size distribution of the material which is to be protected (the base material). The impervious core material is the base with respect to the adjacent filter material which in turn is the base for the transition material to the adjacent rockfill zones.

A classical set of criteria for the grain-size distribution of filter materials is presented in Table 2. 1.

Table 2.1 Criteria for grain-size distribution of filter material

Filter material:	Uniform base material: $\frac{d_{60}}{d_{10}} < 1.5$	Well graded base material: $\frac{d_{60}}{d_{10}} > 4$
Uniform filter $\frac{D_{60}}{D_{10}} < 1.5$	$5 < \frac{D_{50}}{d_{50}} < 10$	$5 < \frac{D_{50}}{d_{50}} < 15$ $\frac{D_{15}}{d_{85}} < 5$
Well graded filter $\frac{D_{60}}{D_{10}} > 4$	$4 < \frac{D_{15}}{d_{15}} < 6$ $\frac{D_{50}}{d_{50}} < 25$ $\frac{D_{15}}{d_{85}} < 5$	$4 < \frac{D_{15}}{d_{15}} < 40$ $\frac{D_{50}}{d_{50}} < 25$ $\frac{D_{15}}{d_{85}} < 5$

Remark: The letters, *D* and *d*, represent grain-size (diameter) of filter and base material respectively. Number suffixes give the percentage by weight of the material being finer than the given diameter.

The main purpose of filters is to prevent erosion of the base material on the upstream side of the filter. Fines may be washed out even when the filter is capable of retaining coarser particles. As far as Table 2. 1 is concerned, the following additional requirements apply in Norwegian practice:

For broadly graded soils, only the 0 to 60 mm fraction of the base material shall be considered. For filters, the grain-size distribution shall be determined for the material as a whole.

Thus, the suitability of a material for use as a filter in an embankment dam shall be judged by its grain-size distribution curve as compared to that of the base material. Probable variations must be reckoned with in the base material. As the filter must cover these variations, the upper and lower limits

of acceptable grain-size distributions of the filter will depend on the variability of the base material. If the base material is very variable, the limits of the filter's grain-size distribution will be narrowed. If the variation of the base material is small, the limits of the filter's grain-sizes will be widened.

The filter criteria apply to adjacent zones throughout the dam. Zones with widely varying or narrow gradings will put stricter bounds on the grading of adjacent zones, sometimes necessitating additional zones of intermediate gradings to be placed in the dam. The ideal filter materials for a typical Norwegian moraine would be broadly graded gravelly sand without stones of more than 20-50 mm size.

In the 1970s it was internationally recognized that the core in many dams had cracked leading to concentrated leakage. To prevent erosion due to such leakage Sherard and Dunnigan (1985, 1989) recommended design criteria for filters as shown in Table 2. 2. Their recommendations are based on many years of research and laboratory testing.

Table 2.2 Filter criteria recommended by Sherard and Dunnigan (1985)

Base material:		Filter material: Recommended design criteria including adequate factor of safety
Group	% fines[1] < 0. 075 mm	
1	85 - 100%	$D_{15} < 9\, d_{85}$, but < 0. 2 mm
2	40 - 85%	$D_{15} \leq 0.7$ mm
3	0 - 15%	$D_{15} \leq 4\, d_{85}$
4	15 - 40%	$D_{15} < 0.7$ mm + $(4d_{85}$–0.7 mm)(40-A)/(40–15)

(1) The percentage of fines and d_{85} are determined from the sand fraction (< 4. 75 mm) of the base material.

(2) The *A*-value is the percentage of the base material passing the 0. 075 mm sieve after regrading.

In general the criteria set out in Table 2. 2 are more conservative than those of Table 2. 1. As adequate filters are of utmost importance for the safety of a dam, it is recommended that both criteria should be satisfied for filters protecting the impervious core. Vaughan and Soares (1982) have adopted a somewhat different approach, by showing that if a filter material is to retain particles of a given size, its permeability must not exceed a certain value.

Quarried rock material with a moderate content of larger stones (< 100-200 mm) and fines would normally be well suited for the transition to the adjacent coarser rockfill.

Materials for filters and transitions may also be produced by crushing and screening. For such sharp edged, angular materials the filter criteria presented in Table 2. 1 should be met with ample margins.

2.4 Rockfill Materials

High quality rockfills are essential in rockfill dams with a comparatively rigid and brittle, or especially slim impervious element. It is very important that the impervious element in the dam is not subjected to excessive deformations which may lead to cracking, and this depends on the quality of the rockfill embankment.

As the strength and compressibility of the rockfill depend on grain shape, size distribution and strength of individual rock fragments, it is important to locate the quarry within rock formations which give the best quality of quarry-run rock materials. Granite or gabbroic rock formations, be they granites, diorites, gabbros or gneisses, may yield quarry-run rock of high quality. Shales and schists such as clay shales, mica schists and phyllites yield low quality rock which often do not satisfy the requirements, particularly for the exterior protective covers of the dam. Weathered zones must be avoided in any rock formation. As a general rule, any rock with a compressive strength of 30 MPa or more is satisfactory (Cooke and Sherard, 1987).

The protective cover of the upstream slope consisting of selected large stones of sufficient weight to withstand wave action, must be resistant to frost and weathering forces (Section 6.7). Stones for the protective crown cap and the toe of the downstream embankment are usually of a size in the order of 1 m^3.

The degree of fissuration in the rock mass and the method of blasting will decide what size of stones results from the quarrying. Distances between joints in the rock mass must at least be equal to the equivalent diameter of the required stone size. Microfissures, for instance in limestones, can render the production of block-size material impossible. This may often be hard to detect without test blasting in the actual quarry.

Quarries, in disputable cases, must therefore be thoroughly investigated through geological mapping, blasting tests and laboratory testing. The location should be chosen giving due consideration to quality, technical, environmental and economic aspects.

2.5 Borrow Pit Investigations

The overburden at prospective borrow areas must be examined with respect to its depth and type. Similar examinations are required when substantial overburden exists at the dam site. If the site is readily accessible, and the overburden is thought to be less than 3 to 4 m thick, investigations are best carried out by digging test pits.

If the site is inaccessible to excavation equipment, or if the thickness of overburden is greater, seismic refraction profiling is preferable as an initial step in the investigations. Seismic refraction profiling consists of measuring the propagation velocity of compression waves through the ground. Measurements are made on straight line profiles along which geophones are placed at regular 5 to 10 m intervals. Small explosive charges are placed along the same profile line at 30 to 50 m intervals and detonated one at

a time. The ground shocks from each detonation, propagating through the various layers of the sub-surface, are picked up and recorded by the geophones, measuring the time elapsed between the detonation and arrival of incident ground waves. As propagation velocity normally increases with depth, the velocity in each subsequent layer can be determined, and their thickness and the total depth to bedrock be estimated.

The propagation velocity of ground compression waves increases with increasing resistance to dynamic deformations and decreases with increasing density of the medium. For example the compression wave velocity of water is about 1500 m/s, whereas for ice it is 3500 m/s.

At constant porosity, the density of a porous material will increase when the pores are filled with water. The reduction in propagation velocity due to increase in density is, however, more than offset by the increase in velocity due to the higher degree of saturation, which increases resistance to dynamic volume changes. Propagation velocities are therefore much higher below the ground water table than they are in less saturated parts of the same stratum. Similarly, consolidated sediments exhibit higher wave propagation velocities than looser sediments. Figure 2. 2 shows the normal range of compression wave propagation velocities for various materials above and below the ground water table.

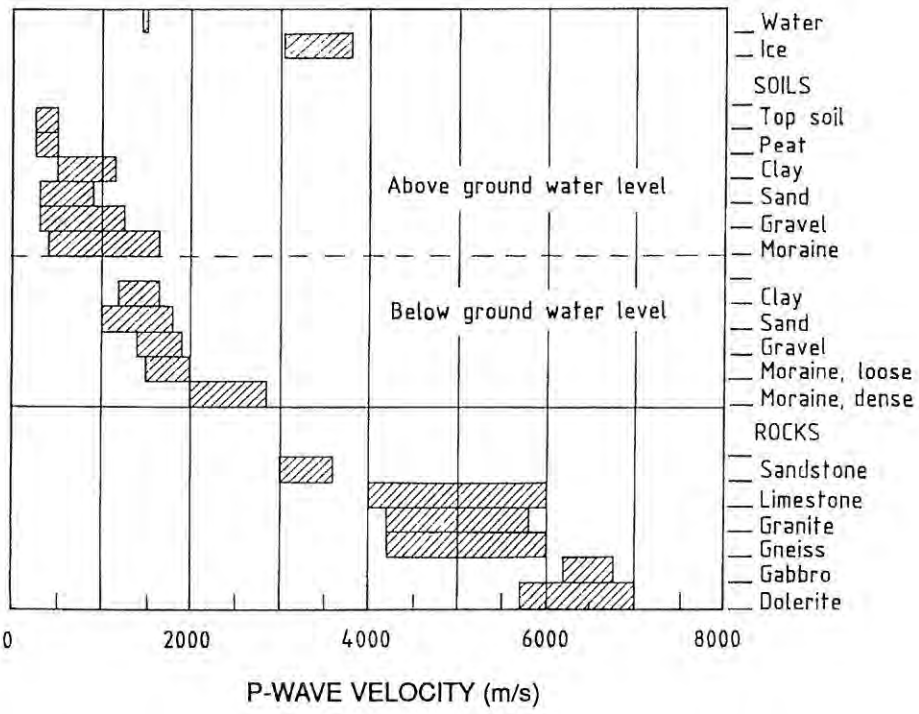


Fig. 2.2
Compression wave propagation velocities for various materials

The difference in wave propagation velocities in loose overburden and in rock is normally so great that the rock surface can be determined fairly accurately, especially when the surface is regular and even. If, however, deep and narrow clefts or gorges with loose fillings or steeply dipping faults or crushing zones cut through the rock mass, it is very difficult if not impossible to determine which of these irregularities is present.

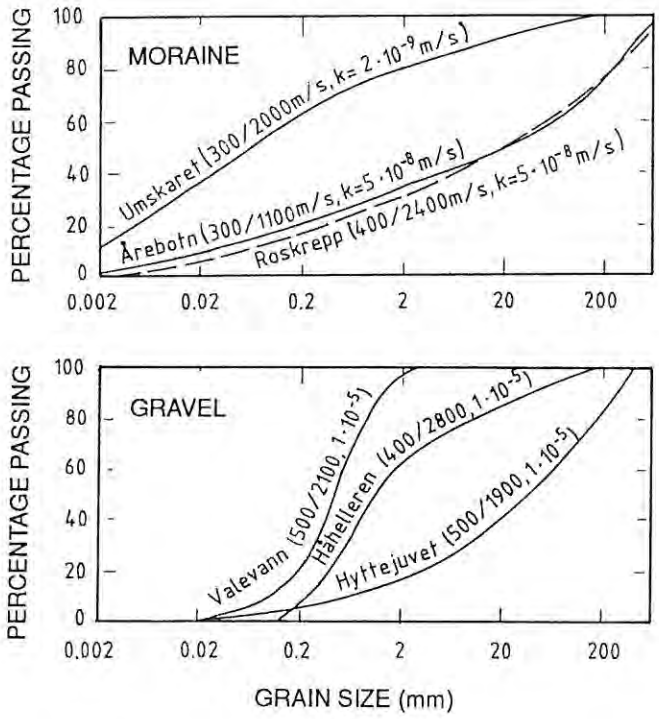


Fig. 2.3
Compression wave propagation velocities measured above/below the ground water table in moraine and gravel deposits with known grain-size distributions and permeabilities

Figure 2. 3 gives examples of compressional wave propagation velocities in moraine and gravel of various gradations, and confirms that seismic refraction profiling alone cannot distinguish gravels from moraine.

Although seismic methods offer excellent techniques for exploring underground conditions, supplementary testing through borings or test pits is usually needed. Drilling methods and equipment must be adapted to penetrate the actual ground conditions encountered and to fit the purpose of the hole which may include sampling of both sediment and underlying rock as well as ascertaining the exact position of the rock surface.

Rock sampling is commonly done by coring, requiring the use of rotary-drilling with hollow cylinder diamond studded drill bits. Comparatively heavy drilling equipment is needed to penetrate moraine and gravel, and borings in such materials require steel casing driven down in stages and retrieved for reuse.

Sampling to shallow depths can be achieved by digging test pits. Samples are taken from different levels in the side walls of the pit. The samples are registered and their location marked on the map and one cross-sectional profile of the area of interest. Photos of the test pit and the excavated material may give valuable information about the character of the bulk material. Sinking shafts by hand digging was practised to depths of 8 to 9 metres as recently as 1970. Today, digging by hand would be out of the question except for very limited and shallow exploratory pits.

For moraine the natural water content should be determined and evaluated for comparison with the optimum water content, as discussed in Chapter 3. The extent to which a potential deposit should be investigated depends on

the magnitude of the deposit as compared to the actual volume needed. If the deposit volume is several times bigger than the volume needed, it may be enough to confirm that it contains material of suitable quality. If, by contrast, the exploitable volume barely covers the need, it must be thoroughly investigated to find out to what extent and under which conditions it will meet the requirements. In practice, otherwise usable materials may be spoiled by soaking, soiling and faulty arrangements in the operation of the borrow pit. It is therefore advisable to keep in mind the "rule of thumb" that the volume of presumed exploitable deposits should be at least twice as large as the net volume needed.

2.6 Environmental Considerations

Environmental concerns and considerations related to hydroelectric power development and dam construction are discussed in a separate volume in this book series (Volume 3).

The field investigations themselves may have adverse environmental effects which by all practical means should be minimized. Special care must be employed at the planning stage of a project, as the initial investigations may affect areas that will not be touched by construction activities at later stages.

Borrow pits and quarries should preferably be located within the future reservoir area so that potential scars in the landscape are avoided. Such solutions must be explored and exploited. Borrow pits within the reservoir may, however, delay and restrict beneficial impoundment during construction, which can be a matter of economic and technical importance (Section 1. 5). Blast craters and test pits should be backfilled as soon as possible, and ignition wiring, markings and miscellaneous litter from the activities must be collected and removed.

3.
PROPERTIES OF COMPACTED MATERIALS

The process of placing and compacting embankment materials is essential as described in Chapter 7. The discussion in this chapter requires some basic definitions and phase relationships which are presented below.

Compacted fill materials are mixtures of solids, water and air. Table 3.1 presents definitions, designations and symbols used internationally to describe the relative contents of solids, water and air. The densities of air and of water are known and are set to be zero and 1000 kg/m^3, respectively. The values of the other parameters are determined by measuring weights and volumes.

The mass of the wet sample, m, and the mass of the solids in the sample, m_s, are determined by weighing before and after the sample has been dried. Hence, the difference between wet and dry mass gives the mass of water in the wet sample:

$$m_w = m - m_s$$

Provided that the specific density of the solids has been determined, the volume of solids can be calculated:

$$V_s = \frac{m_s}{\rho_s}$$

The volume of the air content cannot be determined by weighing or by direct measurement. Presumably, this realization led Proctor in 1933 to introduce the term dry density, ρ_d , which allows the determination of air volume by weighing solids (dry sample) combined with measuring the total volume of the wet sample. On the basis of weights and volumes thus determined, the airfilled pore volume, V_a , the total density, ρ, submerged density, ρ', water content, w, degree of saturation, S_r , air void ratio, A, porosity, n, and void ratio, e, can be calculated as shown in Table 3.1.

3.1 Compaction and Water Content

The purpose of compaction is to reduce pore volume, V_v , of the fill material. This entails a reduction in total volume and hence an increase in the dry density. Proctor investigated the effects of compaction of soil samples by

Table 3.1 Symbols and definitions of various terms used in describing the relative contents of solids, water and air in fill materials

Symbols for mass		Element	Symbols for volume		
m	$m \approx 0$	Air	V_a	V_v	V
	m_w	Water	V_w		
	m_s	Solid	V_s		

Density of air	$\rho_a \approx 0$	Unit weight of air	$\gamma_a \approx 0$
Density of water	$\rho_w = \frac{m_w}{V_w}$	Unit weight of water	$\gamma_w = g\rho_w$
Density of solid	$\rho_s = \frac{m_s}{V_s}$	Unit weight of solid	$\gamma_s = g\rho_s$
Dry density	$\rho_d = \frac{m_s}{V}$	Dry unit weight	$\gamma_d = g\rho_d$
Water content	$w = \frac{m_w}{m_s}$		
Degree of saturation	$S_r = \frac{V_w}{V_V} = \frac{w}{\rho_w/\rho_d - \rho_w/\rho_s}$		
Total density	$\rho = \frac{m}{V} = n\,S_r\,\rho_w + (1-n)\rho_s = (1+w)\rho_d$		
Submerged density	$\rho' = (\rho_s-\rho_w)\,(1-n) = (\rho_s-\rho_w)\left(1 - \frac{\rho_s-\rho_d}{\rho_s}\right)$		
Porosity	$n = \frac{V_v}{V} = \frac{e}{1+e} = \frac{\rho_s-\rho_d}{\rho_s}$		
Void ratio	$e = \frac{V_v}{V_s} = \frac{n}{1-n} = \frac{\rho_s-\rho_d}{\rho_d}$		
Air void ratio	$A = \frac{V_a}{V} = \left[1 - \frac{\rho_d}{\rho_w}\left(\frac{\rho_w}{\rho_s} + w\right)\right]$		
Degree of compaction	$D_p = \frac{\rho_d}{\rho_{dmax}}$		
Relative density	$D_r = \frac{e_{max}-e}{e_{max}-e_{min}}$		

packing earth into a cylinder. His apparatus and procedure have since become international standard, referred to as Proctor test and the Proctor apparatus (Figure 3.1).

The mould into which the sample is packed has a diameter of 105 mm and a height of 165 mm. The cylinder is divided in two parts. When compaction is finished, the upper part of the mould is removed and the sample is cut off leaving the lower part containing a specimen 115.5 mm high and 1.0 litre in volume, for determination of the water content and the dry density of the compacted material. The test mould stands on a rotating base plate, and the sample is compacted with a falling weight with a diameter of 50 mm. By turning the mould between each fall of the weight, the compaction blows are distributed evenly over the sample surface. In the *standard* test the falling weight of 2.5 kg is dropped 300 mm onto the material in the mould.

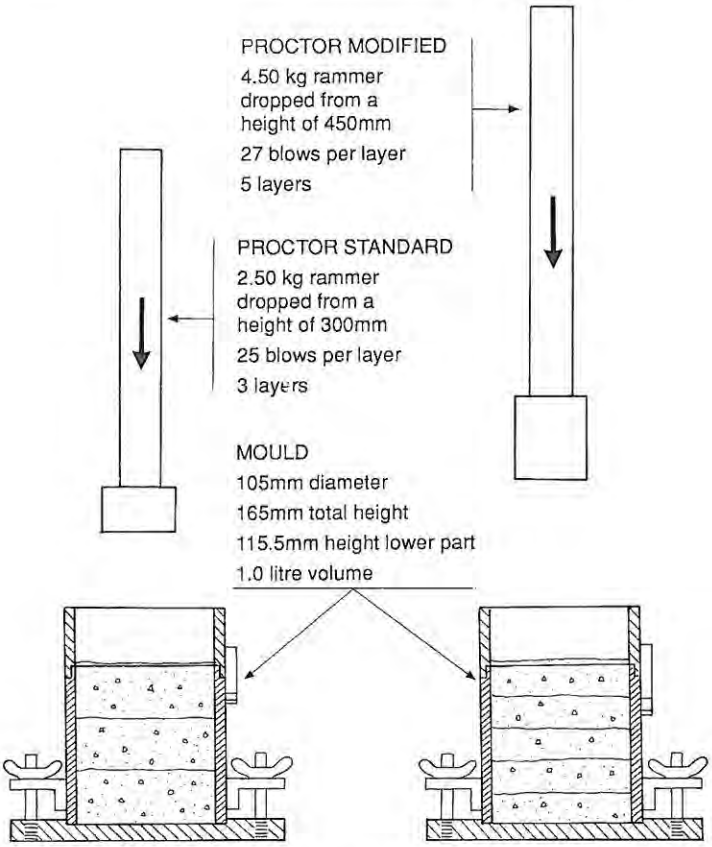


Fig. 3.1
Proctor apparatus for investigating compactability of earth materials

A compaction test requires 12-15 kg of material less than 16 mm in diameter. The material is placed in the test mould in three layers of about 40 mm thickness after compaction. Each layer is compacted by 27 blows of the falling weight. Thus, the specific compaction work for the standard test amounts to approximately $5.5 \cdot 10^5$ Nm/m^3 of compacted material. In the *modified* test a falling weight of 4.5 kg is dropped 450 mm 27 times onto each of the five layers in the mould. The specific compaction work for the modified test amounts to approximately $26.8 \cdot 10^5$ Nm/m^3.

Plotting the results from compaction tests in terms of dry density versus water content shows that the effect of compaction is significantly influenced by the water content in the sample (Figure 3.2).

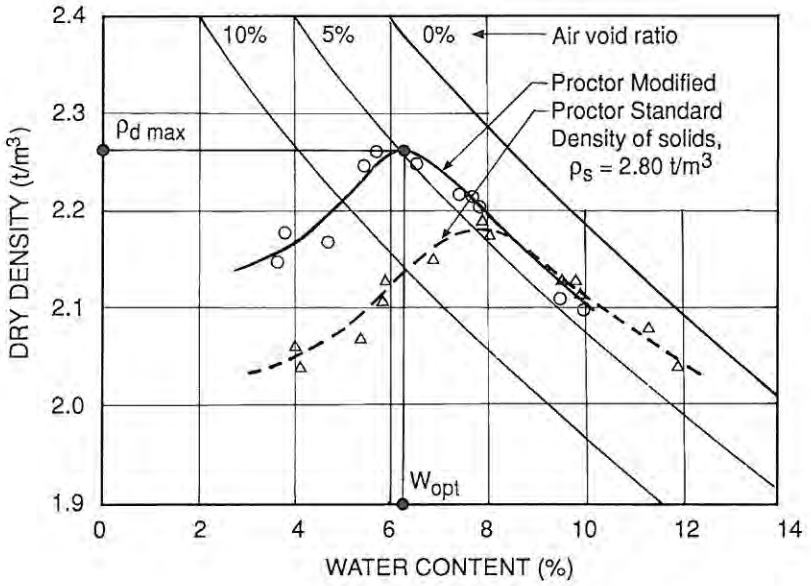


Fig. 3.2
Typical compaction curves for moraine using Proctor tests

When "impervious" soil is relatively dry, separate cohesive lumps of material may be firm enough to prevent them from breaking down and filling the voids between them under compaction. Hence a relatively low dry density will result. When the water content is increased, the soil becomes more plastic and the voids will be reduced during compaction, resulting in higher dry density. If the water content is comparatively high, the pore volume may become saturated. As the water in the pores of fine grained materials cannot be squeezed out by compaction, the total pore volume will remain high. Increasing water content beyond a certain value will, therefore result in decreasing dry density.

In any fine grained soil material subjected to a given amount of compaction, the maximum dry density is obtained at a certain definite water content. This water content is called the optimum water content and the corresponding dry density is called the maximum dry density. Figure 3.2 presents the compaction curves from Proctor standard and modified tests on a moraine. For Proctor standard the maximum dry density 2.18 t/m^3 is obtained for an optimum water content of 7.9%. The higher compaction effort of the Proctor modified test increases the maximum dry density to 2.27 t/m^3 while the optimum water content is reduced to 6.2%. By comparing the compaction curves with the curves for different air void ratios, it appears that at optimum water content the air volume is about 5% for both tests.

The specific compaction effort chosen by Proctor for the standard compaction test corresponds to the effect of 12 to 16 passes with the relatively light sheep-foot rollers used for compaction of earth dams during the early nineteen-thirties. By using heavier equipment, and thus increasing the specific compactive effort, one alters the shape of the compaction curve. How the compaction curve varies with the specific compactive effort may be illustrated by laboratory tests in which the number of blows on each layer in the Proctor apparatus is varied. Figure 3.3 shows how the dry density increases with increasing compactive effort, i.e. number of blows. During the initial stage of compaction the density is increased rapidly. When a certain degree

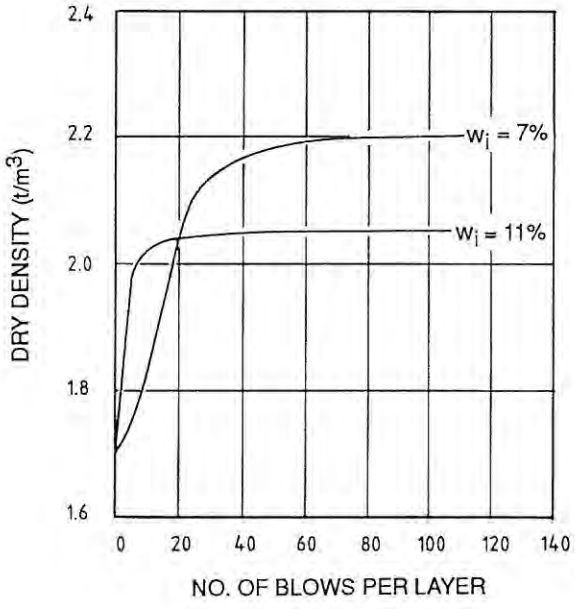


Fig. 3.3
Dry density depending on the amount of compaction effort in the Proctor test. Tests on moraine with two different water contents.

of compaction is reached, further compaction gives only a small increase in dry density.

In the figure, two curves for different water contents, 7% and 11%, respectively, are presented. If sufficient compactive effort is applied, the lower water content gives a significantly higher dry density than the higher water content. The density curve for 7% water content levels out after 60-70 blows, but does so after only 15 blows for 11% water content. With a high water content only a small amount of compactive effort is required to close the existing voids. As water cannot be forced out of relatively impervious materials by the process, continued compaction will not increase the dry density any further.

Figure 3.4 presents the results of a series of tests on moraine for three different degrees of compactive effort. The curves show that the effect of increased compactive effort increases with decreasing water content, and the optimum water content decreases with increasing amount of compactive effort applied.

In the same figure, the dry density obtained from the field compaction of 0.8 m thick layers of the same moraine with 8 passes of an 8 ton vibratory roller is plotted. Compared with the curves from the two Proctor tests, it is clear that the vibratory roller has a compaction effect similar to that of the modified test for water contents less than about 10% or about 4% points above the optimum water content for this test. For higher water content the compaction effect is similar to that of the standard test.

The determination of optimum water content by Proctor tests is primarily of interest for "impervious" materials which will retain water during compaction. No optimum water content can be determined for coarse materials, such as gravel and rock material, where surplus water drains away during compaction. However, the effect of compaction is known to be enhanced when such materials are soaked and thoroughly wet during the compaction process.

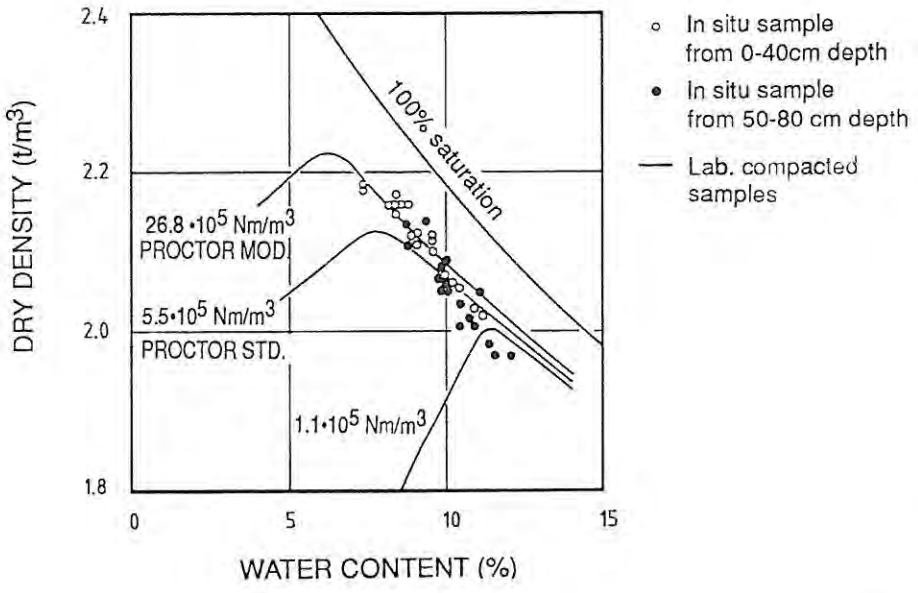


Fig. 3.4
Laboratory and field compaction of moraine. The field results are from 0,8 m thick layers compacted with 8 passes of an 8 ton vibratory roller.

The degree of compaction of coarse materials is not expressed by the dry density, but as relative density:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \cdot 100\%$$

where e: void ratio upon compaction of the material;
e_{max}: void ratio at the lowest possible density of the material;
e_{min}: void ratio at the highest possible density of the material.

The relative density thus ranges from 0 to 100%.

No standard method exists for determining maximum and minimum density of granular materials. With some materials maximum density is achieved by tamping, with others by vibration. The minimum density, especially that of comparatively fine grained materials, is determined by measuring volume and weight of dry material loosely poured into a glass cylinder (Bishop and Henkel, 1962; Berre, 1981).

3.2 Permeability Testing

In 1856, Darcy made measurements using the arrangement shown schematically in Figure 3.5. He found that the discharge velocity of water in a laminar flow through a soil sample is directly proportional to the hydraulic gradient applied in the test. Darcy's law states:

$$v = k\, i$$

where v: discharge velocity, defined as, $v = Q / A\, t$ (Figure 3.5);
i: hydraulic gradient, $i = h/l$;
k: coefficient of proportionality, simply called permeability.

The permeability, k, equals the specific discharge velocity at a gradient equal to one.

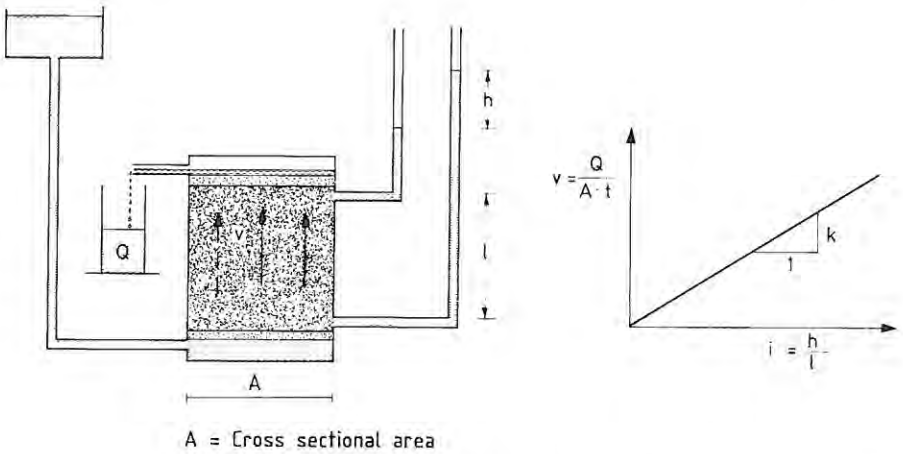


Fig. 3.5 Principle and definitions for determination of the permeability of soil

The permeability of different soil types depends mainly on the grain size distribution of the material, the decisive factor being the size of the individual pore openings and not the total pore volume as such. Wider pores entail higher flow velocities than narrow pores, and the pore sizes will be bigger in coarse grained than in fine grained material. Clay exhibits far lower permeability than sand although the pore volume in clay may be much higher.

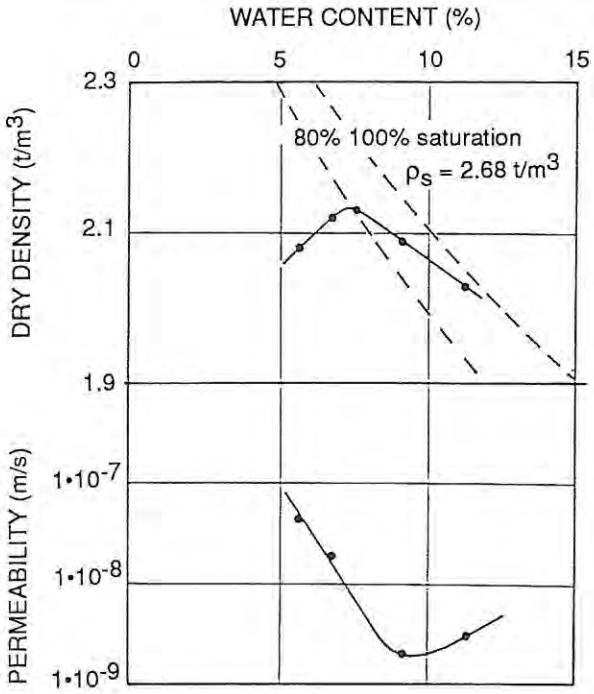


Fig. 3.6
Results of Proctor compaction and permeability tests on moraine

The permeability can be determined by laboratory and in-situ tests. The grain-size distribution of the material is of importance for the type of apparatus to be used for the laboratory permeability tests. Although permeability depends primarily on grain-size distribution of the material, the method of building the test sample into the apparatus will also have an influence. The permeability of impervious material for the core of a dam is determined by Proctor permeability tests where the passage of water through a sample built into a Proctor permeability cell is measured. Typical results of such a test are shown in Figure 3.6. The lowest permeability is obtained with a water content one to two percentage points above the optimum water content.

The porosity of coarse materials, such as gravel and blasted rock, will be reduced by watering and compaction. By reducing the porosity from, for instance, 0.35 to 0.30, the permeability would be reduced by about 30%.

Permeability may roughly be estimated from the grain size distribution. Hazen (1920) established an equation valid for laminar flow through uniformly graded soils:

$$k = 100 \cdot D_{10}^2 \text{ cm/s}$$

where D_{10} represents the sieve mesh diameter in cm through which 10% of the material would pass. For broadly graded materials the Hazen equation overestimates permeability values.

Figure 3.7 shows the relationship between permeability and D_{10} for grain-size distribution curves of different shapes. The permeability estimated by Hazen is indicated by dashed lines in the figure. The measured permeability values invariably come out below the dashed lines due to the fact that the materials are broadly graded and not narrowly graded as presupposed by

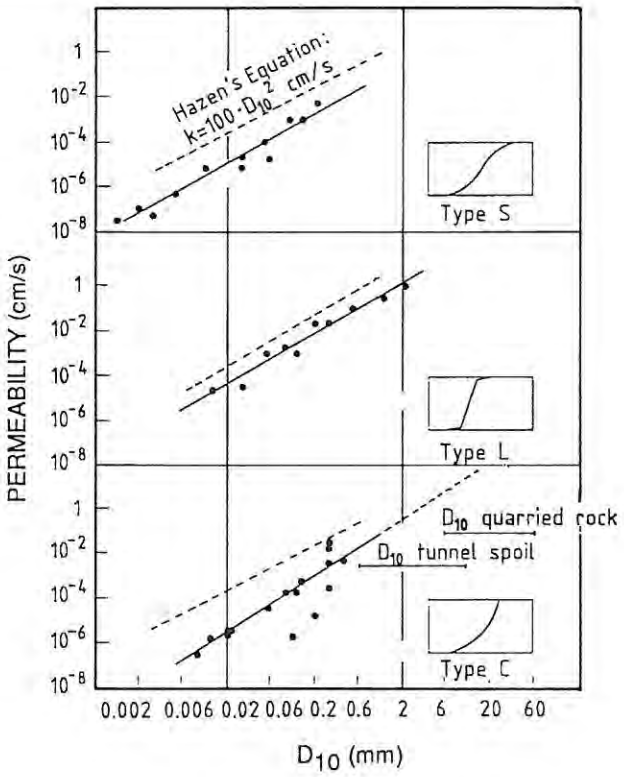


Fig. 3.7 Relation between permeability and D_{10} for typical grain size distribution curves

Hazen. The type L material, which is essentially narrowly graded, therefore shows the least difference between estimated and measured permeability.

3.3 Compressibility and Deformation Modulus

Compression of fills is usually simulated in an oedometer test where the sample is confined within a stiff steel cylinder and the volume change can take place only axially (one-dimensional deformation). An oedometer test is shown schematically in Figure 3.8. The material sample is compacted within the steel cylinder and placed between a bottom and a top filter.

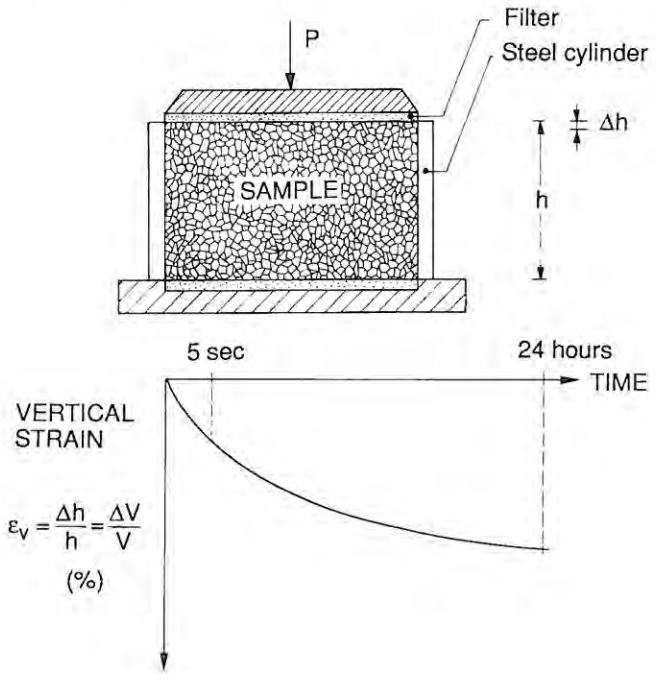


Fig. 3.8 Principle of oedometer tests and registration of results

The load is increased in stages, for instance once a day, and the vertical strain is measured at regular time intervals, for instance, 5, 15, 30, 60 seconds and so forth, after each loading step. (This procedure is described as a load-controlled test. The stress-strain-time relationship may also be determined from deformation-controlled tests.)

Following each loading step the development of vertical strain over time is observed and recorded. During compression the pore volume containing air and water is reduced. The volume of solids does not change, but the individual particles shift positions, however slightly, and cause the pore volume reduction. Hence equilibrium is reached when the individual solid particles have acquired a stable position under the actual load conditions, but creep strains may continue for a long time. In this connection "particles" denote clay minerals in a moraine as well as stones in a rockfill.

As mineral particles slide against each other or rotate, sharp edges and corners will break off or be crushed, causing further movement of particles by a sort of chain reaction in the sample. The speed with which volume compression takes place depends partly on the chain reaction mentioned and on how quickly water can be squeezed out of the sample. The air in the pores will not affect the compression perceptibly, as air is highly compressible. Water, in contrast, is practically incompressible. If the pores are saturated, water must be forced out to allow any reduction of the pore volume. For finely grained materials, such as clay and moraine, permeability will mainly determine the form of the time-compression curve or the consolidation time.

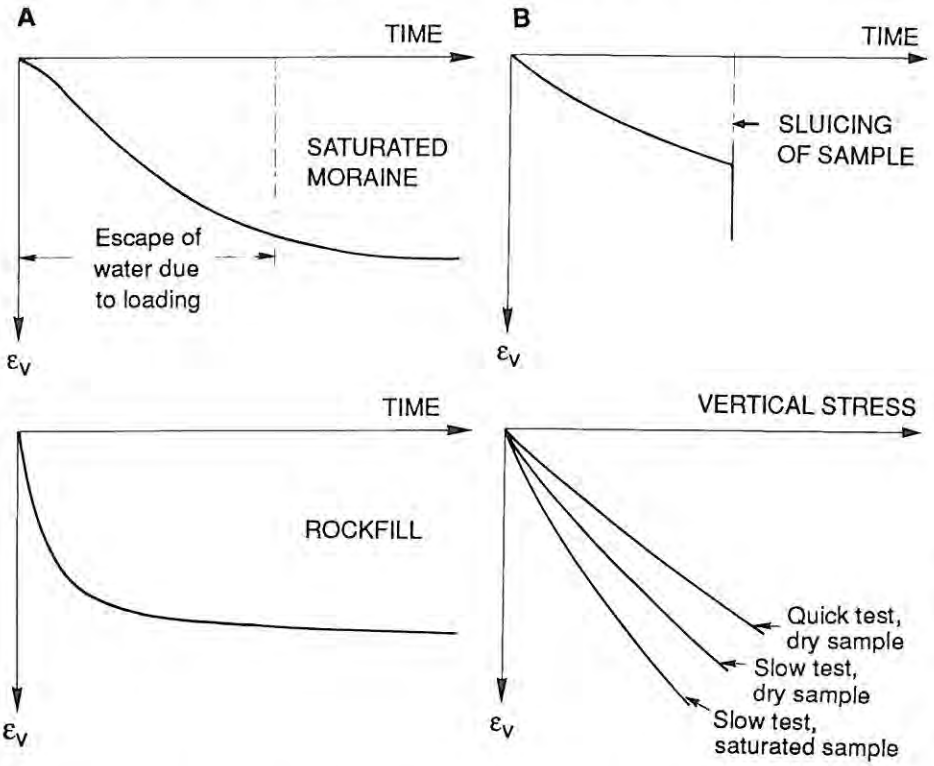


Fig. 3.9
Time-compression curves and effect of soaking
A) Results for a saturated moraine and rockfill
B) Effect of soaking dry rockfill

Figure 3.9a gives examples of the time-compression curves for a saturated moraine as compared to a rockfill sample for a vertical stress increment. The water enclosed in the pores of a fine grained material will slow down

the rate of compression but will simultaneously "lubricate" the particles bringing about a greater volume reduction than would be the case with a dry sample. This "lubrication" effect of water is clearly demonstrated when dry samples of moraine or rockfill are submerged in water under a constant load as shown in Figure 3.9b. Besides the lubricating effect of the water, the dry particles lose strength when soaked. This entails increased breakage and crushing at the points of contact, especially in sharp edged materials like blasted rock.

Figure 3.10 illustrates in a very simplified manner, three possible alignments of brick shaped particles in a granular material matrix, and how compression of these samples under equal loading would be essentially different. Sample A would be practically incompressible, and sample B would show little change in volume until the load starts breaking individual brick particles in the matrix. However, sample C would allow slipping and breaking of edges and rotation of bricks, leading to collapse and a rapid reduction of pore volume even at low loads.

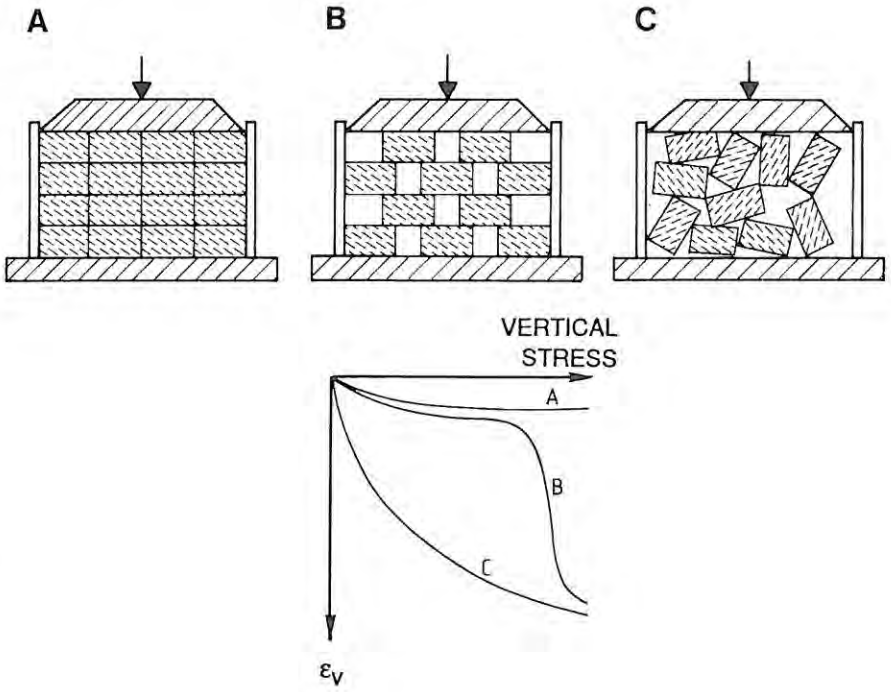


Fig. 3.10
Compression of samples with different matrix configurations

Although of equal initial porosity, sample C exhibits a much higher compressibility than sample B as long as individual bricks do not crumble completely under the actual load. If the voids of sample C were filled with smaller stones, the initial pore volume would be less and the particle matrix more rigid, resulting in a lower compressibility.

The structure of the material sample built into the oedometer is clearly of importance. For two samples of the same material compacted to the same porosity, it appears that vibratory compaction results in a less compressible structure than what is achieved with compaction by static loading. The structure becomes more stable by "shaking" the particles into close contact rather than by squeezing them together by static pressure.

On the basis of oedometer test results it can be concluded that compression of a compacted sample is essentially a consequence of structural change and local crushing at interparticle contacts. The structure of compacted fills will mainly depend on the size and shape of the constituent particles and how the material is placed and compacted. Well graded material with rounded particles will be less compressible than sharp edged and flaky material of narrow grading. Mineral strength and particle shape affect the extent of crushing at interparticle contacts, and rounded grains of hard, sound rock are more resistant to compression than are sharp edged grains of weaker types of rock.

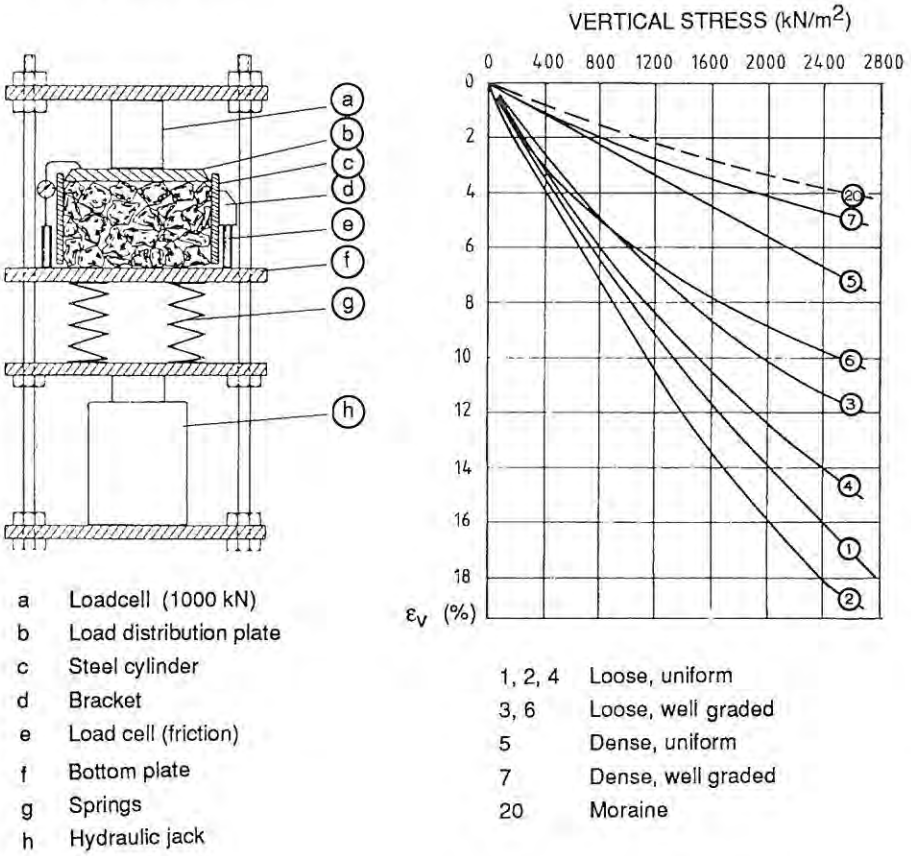


Fig. 3.11
Schematic drawing of an oedometer and results of tests on crushed syenite

Figure 3.11 shows an oedometer of diameter 600 mm for testing of coarse materials and the results of a series of tests. The resistance to compression (one-dimensional deformation modulus) is highest for well graded, compacted material (curves 20 and 7) and lowest for uniformly graded, loose material (1 and 2).

The one-dimensional modulus increases with increasing load. The increase is greatest for well graded loose rockfill (3 and 6) where the modulus is 3-4 times higher for the last loading step than for the first. For moraine (20) and uniformly graded rock material (4) the modulus is doubled from the first to the last loading step of the test.

Field measurements on existing dams prove that the deformation modulus of rockfill can be some ten times higher for properly compacted fills placed in layers of moderate thickness than in high lift, dumped fills (Chapter 5).

3.4 Shear Strength of Fill Materials

The strength of soil and rockfill expressed in terms of shear stress, is given by the Mohr-Coulomb equation (envelope):

$$\tau_f = c' + (\sigma - u)_f \tan \phi'$$

where τ_f : shear strength (shear stress at failure on the failure plane);
c' : shear strength parameter, cohesion, with respect to effective stresses;
ϕ' : shear strength parameter, angle of friction, with respect to effective stresses;
σ : total normal stress on the failure plane;
u : pore pressure on the failure plane.

The effective normal stress, $\sigma' = \sigma - u$, represents the stress transmitted through the points of contact between the particles, whereas the pore pressure is the neutral pressure of the water confined in the voids between the particles. The pore pressure consists of a constant component due to hydrostatic head or a steady state flow, plus an excess, transient component (positive or negative) due to alteration of stress conditions through compaction or changing external loads.

During compaction and loading of fine grained materials such as clay and moraine, excess pore pressures may develop and will take a long time to dissipate. During compaction and loading of coarse gravel and stones, water will easily escape without any excess pore pressures being created. These materials are therefore referred to as free draining.

As will be discussed below, the shear strength parameter, tan ϕ', usually decreases with increasing effective stress level. Thus, the shear strength envelope does not represent a straight line in a Mohr-Coulomb diagram. The equation is, however, approximated by a straight line over the range of effective stress which is of interest in each case. The shear strength parameter, c', should, therefore, not be considered as a physical adherence between particles, but rather be taken simply as the intercept of the straight line with the τ-axis of the Mohr-Coulomb diagram (Figure 3.12).

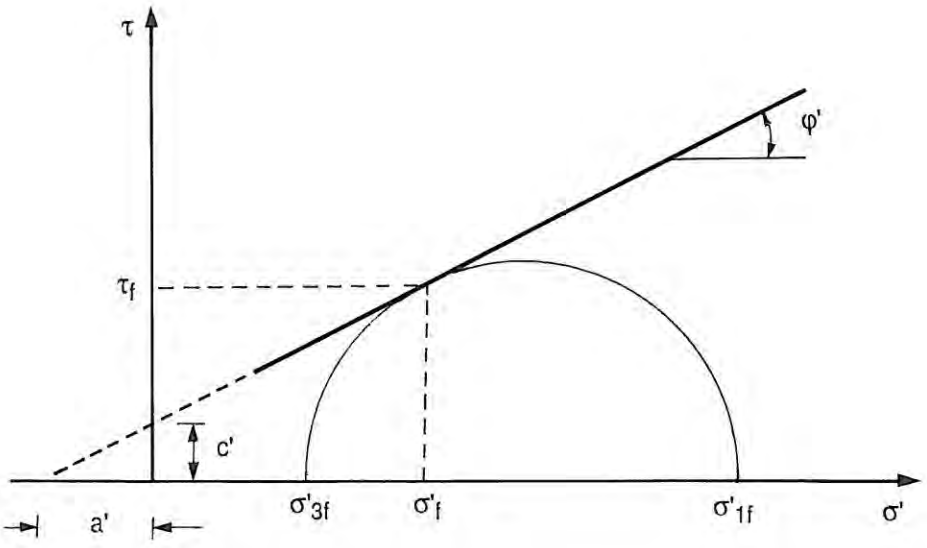


Fig. 3.12 Mohr-Coulomb representation of shear strength parameters

If the equation for shear strength τ_f is transformed by using the principal stresses at failure, σ'_{1f} and σ'_{3f}, the following relationship is obtained:

$$\sigma'_{1f} = N_\phi \sigma'_{3f} + 2c' \sqrt{N_\phi}$$

or

$$\sigma'_{1f} = N_\phi \sigma'_{3f} + a'(N_\phi - 1)$$

where

$$N_\phi = \frac{1 + \sin \phi'}{1 - \sin \phi'} = \tan^2 (45° + \phi'/2)$$

$$a' = c' / \tan\phi'$$

Fine grained material contains fines popularly called binder. When moist, such material may be moulded into balls, similar to forming balls of wet snow. In addition to the effect of the binder, capillary forces contribute to keeping the fine grained material together.

Stone materials have no binder or capillarity. Dumped into a slope the material surface will attain a certain inclination called the natural slope angle. The natural slope angle is approximately equal to the angle of friction for the material in a loose state. If, however, a shallow ditch is excavated in such material, it may be noticed that the ditch walls can stand almost vertically. This effect is attributed to interlocking of the individual stones, a dry masonry effect which exists notably at very low stress levels. The structure of the material thus has an influence on the shear strength much in the same way as it influences the oedometer deformation modulus discussed in the previous section.

The shear strength of compacted materials is usually determined by means of triaxial tests (Figure 3.13). Berre (1981) presents a comprehensive discussion of triaxial testing techniques and procedures. The apparatus and procedures used depend on whether the samples to be tested consist of impervious "cohesive" material, like clay, or of free draining materials like sand, gravel or stone. A compacted sample of clay is contained in a thin

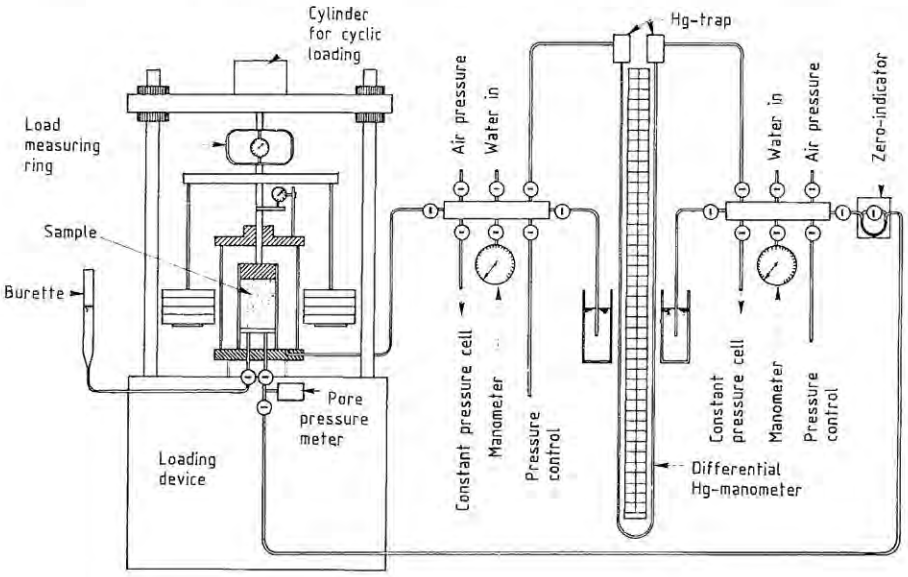


Fig. 3.13
Triaxial apparatus
(After Berre, 1982)

rubber membrane tightly attached with O-rings at both ends to the top- and bottom steel plates. The end plates are fitted with filters connected to the piping and measuring units which are saturated with water.

Within the triaxial cell the sample is subjected to an external water pressure (cell pressure) which entails reduction of pore space due to the content of compressible air in the pores in a partly saturated sample. The ensuing pore pressure is measured, often independently for pore water and pore air phase.

The sample is usually brought to failure by increasing the vertical load through a piston. During the test the vertical stress at the top of the sample, σ_1, the external water pressure acting at the outer surface of the rubber membrane, σ_3, the pore pressure, u, the vertical strain, ϵ_1, and the change in volume, ΔV, are measured. Testing in triaxial cells may also be carried out by increasing horizontal stress on the sample or by simultaneously increasing or reducing the vertical and horizontal stresses to the point of failure.

The shear strength of "impervious" materials, where the water in the pores cannot readily escape, will depend on the actual pore pressure developed during the loading process. To be able to predict the load and time dependent pore pressures developing in an earth core, the pore pressure parameter, $\bar{B}$, and the coefficient of consolidation c_v, as defined below, are measured in triaxial tests.

The pore pressure at a point in an embankment prior to any consolidation has taken place, is set equal to:

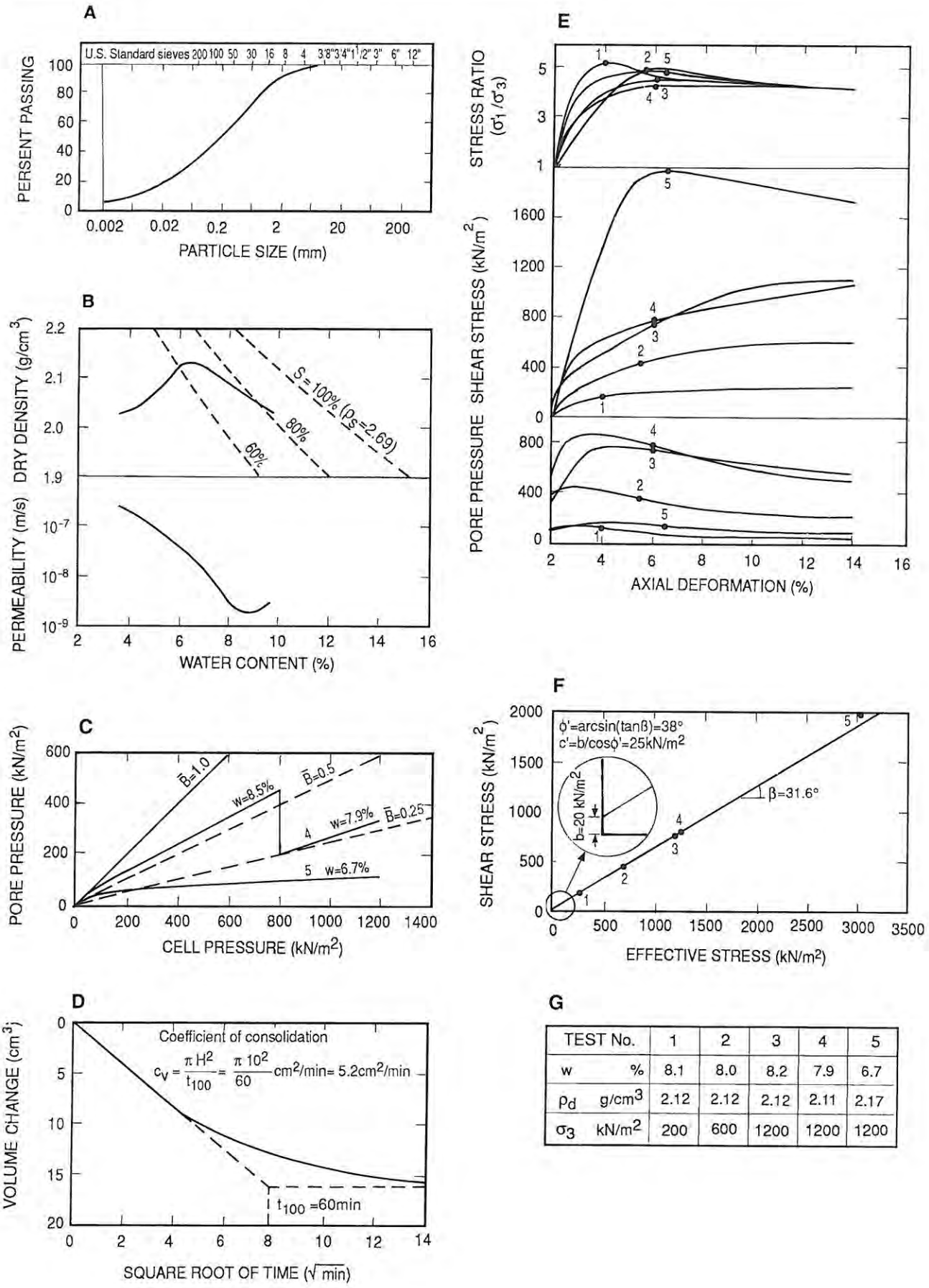
$$u = \bar{B}\ \gamma z$$

where γ: total unit weight of material above the point;
z: vertical depth under embankment surface.

The $\bar{B}$-value is determined in a triaxial test as the ratio of increase in measured pore pressure to increase in axial stress on the sample under undrained conditions. The pore pressure parameter, $\bar{B}$, varies with construction water content and shear dilatancy characteristics of the compacted soil.

The coefficient of consolidation, c_v (dimension m^2/s), characterizes the rate of decrease in pore pressure during consolidation and is proportional to the product of the permeability and the deformation modulus of the material. High permeability and high modulus imply rapid consolidation and high c_v-value. The coefficient of consolidation is determined through oedometer or triaxial tests by measuring the change of volume with time in a sample kept under constant external load during the test and with drainage allowed.

Figure 3.14 illustrates the results of a comprehensive series of tests done on compacted moraine. In addition to shear strength parameters, the stress-strain curves from triaxial tests give the modulus of deformation for different levels of axial strain. This triaxial test modulus is different from the



G

TEST No.		1	2	3	4	5
w	%	8.1	8.0	8.2	7.9	6.7
ρ_d	g/cm^3	2.12	2.12	2.12	2.11	2.17
σ_3	kN/m^2	200	600	1200	1200	1200

Fig. 3.14 Results of laboratory tests conducted on compacted moraine material

one-dimensional oedometer modulus described in Section 3.3. The triaxial modulus decreases rapidly with axial strain as sample failure is approached. Both types of moduli may be specified as tangent or secant values from the corresponding stress-strain curves, and this is the way they are used in finite element analyses of embankment dam deformations (Chapter 5).

With sand, gravel and stones, as opposed to clay, self-supporting specimens cannot be prepared. For this reason and because the specimens, at least for gravel and stone, have to be much larger than for moraine, the tests are performed in a vacuum-triaxial apparatus. The specimen is built into a rubber barrel sleeve which is temporarily suspended within a vertically split steel cylinder. The rubber sleeve is attached tightly to the bottom steel plate and likewise to the top plate after the sample is completely built up. Air and water in the specimen are evacuated by means of a vacuum pump. Atmospheric pressure will thus exert an overpressure all around the specimen, stabilizing it so that the outer steel cylinder bracing may be removed (Figure 3.15).



Fig. 3.15
Vacuum triaxial apparatus in the NGI laboratory

The sample is loaded to failure by increasing the vertical load. During the test vertical stress, underpressure inside the sample as compared to atmospheric pressure, vertical deformation and change in volume of the sample are measured.

The angle of friction, ϕ', in coarse materials depends on grain-size distribution, shape and size of the constituent particles and on the porosity of the specimen. Well graded materials exhibit higher friction than uniformly graded. Sharp edged angular grains give higher friction than rounded grains, and the friction angle will be higher in densely packed specimens than in loose specimens.

The angle of friction is not a pure material property as testing reveals that it is a function of the actual effective stress level. The angle of friction decreases significantly with increasing effective stress in granular materials.

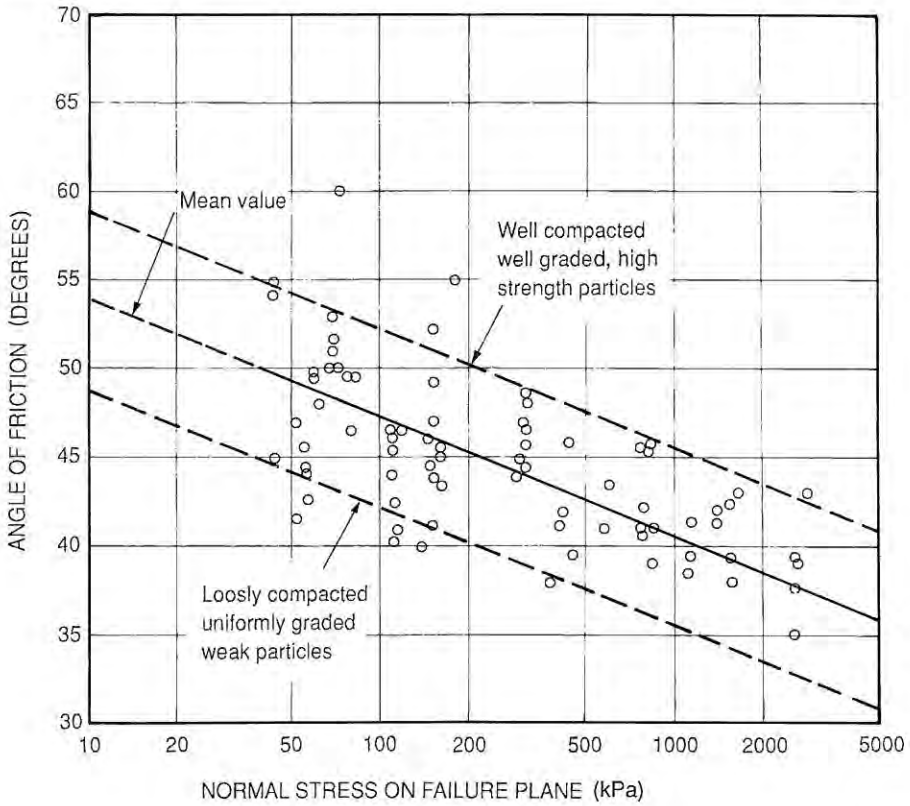


Fig. 3.16
Angle of friction for rockfill as function of effective stress level (After Leps, 1970)

In Figure 3.16 angle of friction in rockfill of different grading and porosity is plotted as function of effective stress on the failure plane.

For dams exposed to earthquakes, the behaviour of the fill materials under cyclic loads must be investigated. Pore pressures, which do not dissipate quickly, may build up during shaking. Lowered shear strength and excessive cyclic and permanent deformations could ensue, rendering the core susceptible to cracking (Section 6.9). Material behaviour under cyclic loads may be investigated by means of triaxial tests and simple shear tests, but properly designed rockfill dams are inherently very resistant to earthquake shaking (Andersen and Høeg, 1991).

4.
LOADS ARISING FROM WATER AND WIND

Water in the reservoir constitutes the basic external load and generates the effects which may inflict damage or at worst, ultimate failure of the dam. In addition come body forces due to embankment weight and potential earthquake loads.

The water head imposes external loads acting on the impervious element, and submerged fills are subjected to uplift. Flow through the dam and adjacent ground sets up pore pressures and seepage forces acting internally, and waves exert loads on the upstream slope of the dam.

4.1 Seepage Forces and Pore Pressures

Versatile finite element modelling and numerical analyses may be applied to compute steady state and transient pore pressures and seepage forces in an embankment dam and its foundation (Pinder and Grey, 1977). However, so far in design practice, the method of "flow nets" has mainly been used, as even an approximate flow net may give the designer a good appreciation of the flow situation.

A flow net is a two-dimensional, graphic representation of the hydraulic condition and is composed of two sets of curves, equipotential lines and flowlines. Equipotential lines designate points of equal potential. For laminar flow the potential at any point is the sum of the elevation of the point above a reference level and the actual pore pressure at the point, expressed in terms of the piezometric height, i.e. the water column above the point. Hence, the equipotential lines join points in the profile for which the piezometric elevation is the same (Figure 4.1).

The flowlines depict the direction of flow paths through the profile. The area between two adjacent flowlines represents a flow channel. A flow net may be produced on the basis of analytical computations, by means of physical models or analog computers or, most often, graphically by hand sketching. The computer can also convert numerical values from finite element analysis into graphical flow nets, simplifying the interpretation of results (Christian, 1980).

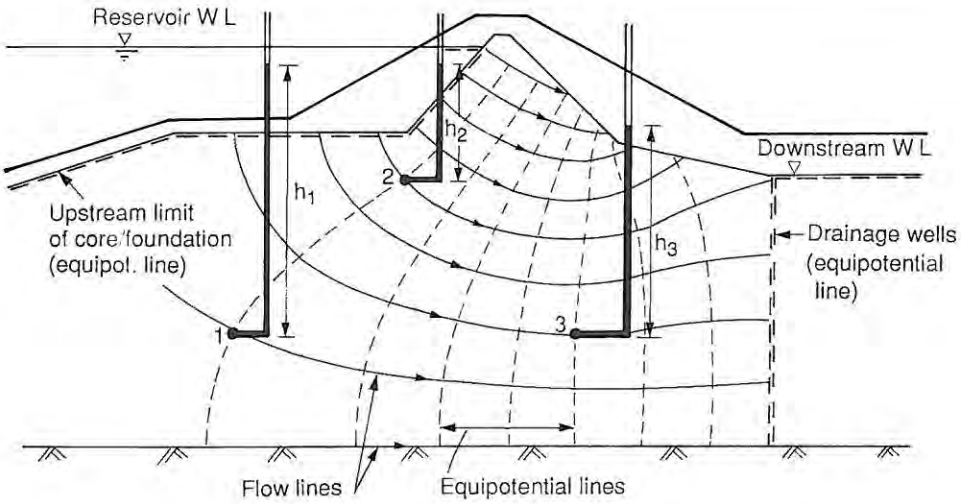


Fig. 4.1
Typical flow net for an embankment dam. Core and foundation are assumed to have same permeability properties.

Under isotropic conditions, i.e. same permeability in all directions, the equipotential lines and the flowlines will intersect each other orthogonally. In practice, the flow net is formed such that the drop in potential from every equipotential line to the next is constant, and each flow channel transports an equal amount of water. These preconditions imply a constant ratio between length and width of each "rectangle" in the flow net (Figure 4.1).

When drawing flow nets, it is first essential to fix the equipotential lines and flow lines given by the boundary conditions. Thus, in Figure 4.1, the flowline along a boundary of watertight rock as well as the equipotential line along the upstream boundary of the impervious core, are given. Likewise, the downstream drainage well which is under hydrostatic pressure, represents an equipotential line. The fact that flowlines and equipotential lines in Figure 4.1 appear in equal numbers is purely incidental. For a more comprehensive introduction to the concept of flow nets the book by Cedergren (1989) is recommended.

By definition, the potential and pore pressure at any point in the section can be determined from the flow net. Hence, the average gradients of flow from one point to another inside the mesh can be determined by measuring the distance between them. The average gradient is defined as the ratio of potential difference to corresponding flow path length.

The drop in potential corresponds to the energy lost over the flow distance between the points. This energy is transferred by friction to the particle matrix through which the water flows, generating a so-called seepage force acting in the direction of the flow. The seepage force per unit volume of the particle matrix is proportional to the flow gradient and the unit weight of water:

$$j = i \cdot \gamma_w$$

where i: hydraulic gradient (dimensionless)

γ_w: unit weight of water

The seepage force exerts an effective pressure in the direction of the flow-line and creates internal stresses that must be added to the effective stresses of the no-flow hydrostatic situation. This is an important principle described for a simple case below.

Figure 4.2 illustrates a situation with vertical, upward exit flow downstream from the embankment toe. In a point at depth, z, below the ground surface a pore pressure, $u = \gamma_w (z + \Delta h)$, exists. The free water level is assumed to be at ground level. The potential drop over the distance, z, equals, Δh, and the average gradient $i = \Delta h/z$.

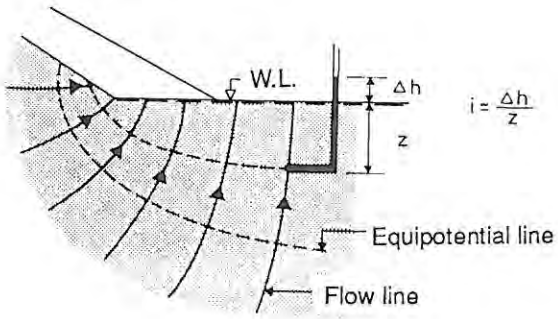


Fig. 4.2
Example of flow net determination of exit gradient. To determine the effective stress situation, use submerged unit weight combined with seepage forces or total unit weight combined with pore pressures determined from flow net.

The vertical effective stress at the same point in the no-flow hydrostatic state is, $\gamma' z$, where, γ', is the submerged unit weight of the material. As the seepage force here acts in the opposite direction of the gravity force, it must be subtracted, and the resulting effective stress at the point in the state of flow becomes:

$$\sigma' = \gamma' z - i\gamma_w z = \gamma' z - \gamma_w \Delta h$$

Alternatively, the effective stress may be calculated as the difference between total stress and pore pressure at any point:

$$\sigma' = \sigma - u = \gamma z - \gamma_w (z + \Delta h) = \gamma' z - \gamma_w \Delta h \quad \text{(q.e.d.)}$$

From this equation, the so-called critical vertical gradient, i_c , at which effective stress at the point would be zero, and at which material particles would become suspended in water, is determined:

$$i_c = \frac{\gamma'}{\gamma_w}$$

To prevent this unstable situation from occurring, the ground surface downstream should be loaded with an inverted filter which increases the effective stresses beneath the surface and stops the erosion and piping. This was the case for the Mänika Dam described in Section 1.3.

From a flow net pore pressures and seepage forces needed for embankment deformation and stability analyses, can be determined. The quantity of seepage through the core can also be calculated by means of a flow net as described in Section 6.4.

4.2 Outflow Forces at Dam Toe

Under normal operating conditions the downstream supporting fill and toe of the dam will not be exposed to seepage forces, and often no requirements are specified for the drainage capacity of the dam. Ample drainage capacity is, however, essential for the safety of the dam if overtopping or big leaks should accidentally occur. This situation must be studied and constitutes a part of the overall risk analysis for the dam.

Outflow of water at the toe of the dam is sketched in Figure 4.3. Depending on run-off intensity, the water may exit on the slope at a certain level above the base determined by the permeability of the fill. The fill is subjected to seepage forces and pore water pressures which reduce the stability along potential sliding planes inside the fill. The overflowed part of the slope is further exposed to surface unravelling and erosion stone by stone.

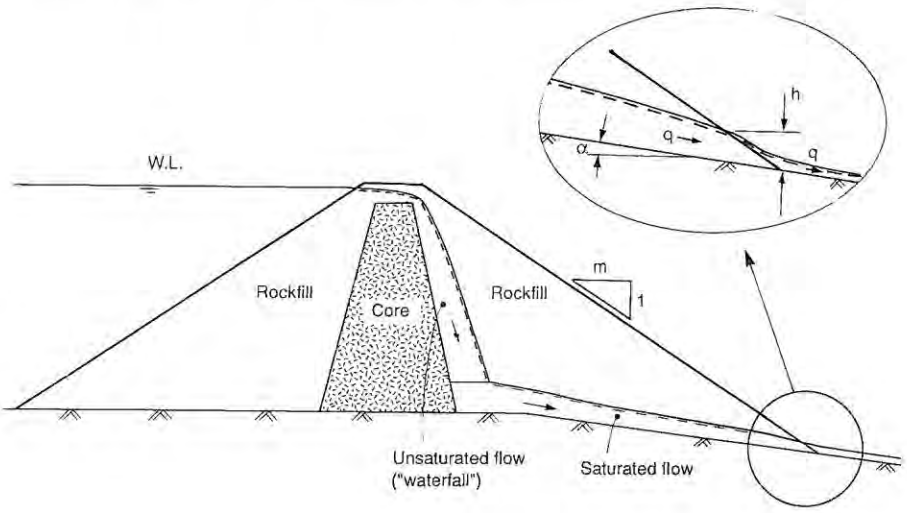


Fig. 4.3
Outflow of water at the toe of the dam. The flow gradient, i, can be estimated by means of even rather approximate flow nets.

Flow through coarse rock fills will generally be turbulent. The flow velocity is calculated as being:

$$v = \sqrt{k_t \, i}$$

where v : discharge velocity;
k_t : permeability at turbulent flow (e.g. m^2/s^2);
i : hydraulic gradient (dimensionless).

The turbulent permeability may roughly be estimated from the equation:

$$k_t = \frac{1}{\beta_0} \frac{n^3}{1-n} \, g \, d_t$$

where β_0: grain shape factor ($\beta_o = 3.6$ for quarried rock);
n : porosity of the fill;
g : acceleration of gravity; and
d_t : significant particle size.

In well graded materials the significant particle size is approximated by $d_t = 1.7 \, d_{10}$. For narrowly graded materials is similarly used, $d_t = d_{50}$. The

sieve opening diameters, d_{10} and d_{50} , represent the sieve openings through which 10% and 50% respectively of the material would pass (by weight).

The porosity, n, may be in the range 20-30% for well graded and well compacted material and 35-40% for narrowly graded material. The permeability, k_t (m^2/s^2), would be in the order of 0.1 d_{10} (m) for the rockfill supporting shoulder and 0.2 d_{50} (m) for the cover stones in the toe and the slope protection. The permeability of the rockfill shoulder can easily be less than 1% of the permeabilities of the protective cover layer of large stones, and will be substantially reduced with increasing content of fines.

The water exits in the slope at an elevation, h, given by the cross-sectional area required for the actual flow (Figure 4.3):

$$h = \frac{q}{\sqrt{k_t i}} \frac{1}{(1 - m \tan \alpha) \cos \alpha}$$

where h : elevation above the reference level;
q : specific flow of water (e.g. m^2/s);
m: slope inclination ratio, $m = \tan \beta$; and
α : slope angle of rock surface in direction of the flow.

The surface of the dam above the tip of the toe and below elevation, h, will be overflowed. The specific flow, q, has a threshold value for stability of the slope depending on the size and weight of the stones, the foundation inclination, α, the effective angle of friction, ϕ', and the slope inclination ratio, m. For a given total outflow, Q, the specific flow, q, will be determined by the shape and width of the dam at the toe. The specific flow may increase dramatically by narrowing gorge effects towards the toe.

The stone sizes required for toe and slope facing, and the critical flow intensity, q, are estimated by means of the empirical equation and diagram presented in Section 6.6.

4.3 Wind Generated Waves

Characteristics of the waves on the reservoir are needed for design of slope protection and freeboard. Wave height and steepness are input data to design rock armoured slope protection against wave erosion. Wind tide and wave run-up are input data required for design of freeboard allowance above maximum flood water level. Waves generated by wind will vary in height even if the wind is of constant force and direction. An example of how the wave height and period vary within a wave train and the corresponding probability density function are given in Figure 4.4.

The irregular motion of wind generated waves is illustrated by the probability density function for the wave height in Figure 4.4b. The motion is characterized by:

H_s: significant wave height; and
T_P: peak period.

The significant wave height is here defined as the mean height of the 1/3 highest waves in a wave train. The peak wave period is the period for a wave corresponding to the peak of the wave height probability density function. The relationship between heights and periods are given in Figure 4.4c.

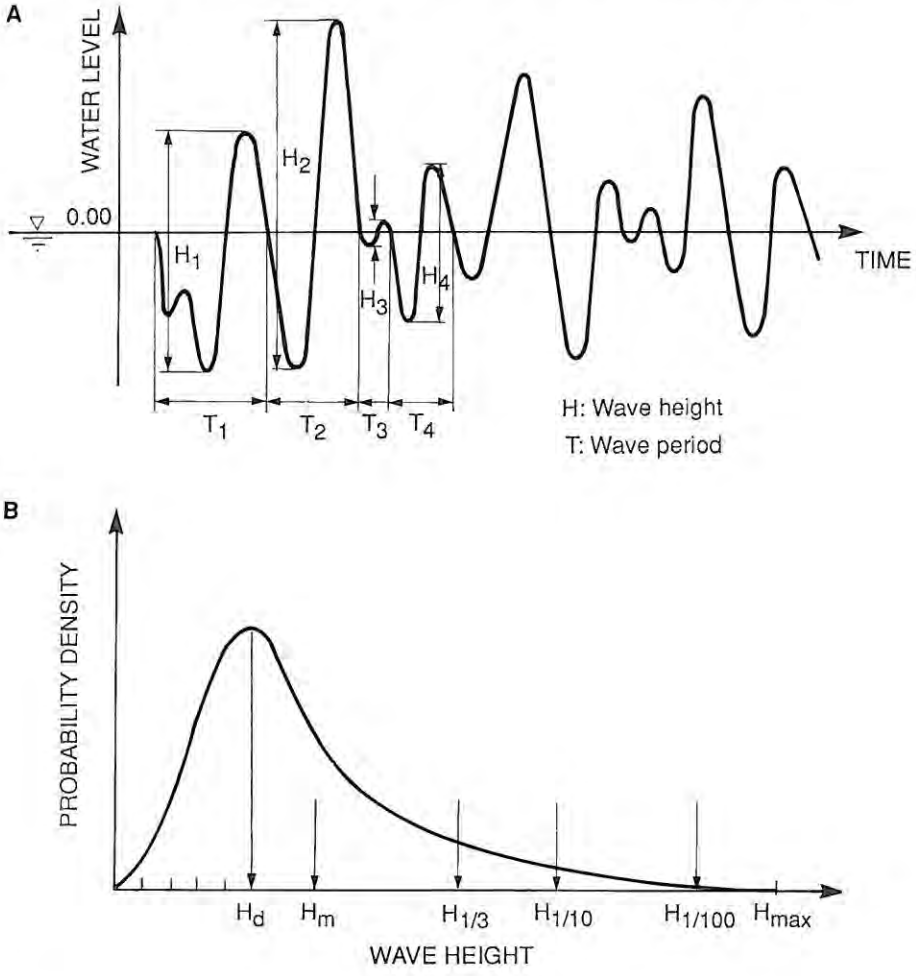


Fig. 4.4
Characteristics of wind generated waves
A) A typical train of waves
B) Probability density function for wave height
C) Statistical relationships for waves based on Rayleigh distribution (After Pilarczyk, 1990)

At the design stage the wave height and period will have to be calculated from observations of the mean wind velocity and duration blowing over the reservoir fetch in front of the planned dam. The governing parameters are:

U: average wind velocity,
t: duration of the wind, i.e. the period over which the wind velocity is averaged, and
F: fetch, i.e. the overwater distance from the shore opposite of the dam in the direction the wind is blowing.

Wind observations are rarely available at a dam site. In most cases, however, the wind regime may be estimated from observations at meteorological stations in the vicinity. If wind occurs only in gusts of a few minutes, this will limit the height of the generated waves. Due to insufficient data the calculation of wave height is usually based on the assumption that the average wind is blowing long enough for the waves to be fully developed. The limitation of the fetch on most reservoirs yields fully developed waves for wind duration between 15 minutes to 2 hours. Thus, for establishing the wind regime the extreme values of the wind velocity for a duration of up to 2 hours will need to be determined.

In many countries the national meteorological bureau has published wind maps, showing wind velocity and direction that can be used to estimate the wind regime at dam sites. An example of such maps for the coastal regions of Norway are given by Børresen (1987). If no such maps are available, the wind at the dam site may be correlated with the wind at meteorological stations in the vicinity. Simultaneous observations at the dam site and at the meteorological station for a certain period allow a transfer function to be established. The transfer function must account for the topographical changes due to the dam structure and filling of the reservoir as smoothening of the terrain reduces surface friction. The procedure for such a correlation is given in Manual (1991).

The wind velocity for design of slope protection and freeboard can be made for a return period of 50 years. In Norwegian practice a steady and lasting wind of 30 m/s has been assumed when no observations are available at the site. In projects where wind effects become important for design decisions, wind observations should be performed at the dam site.

Most reservoirs are narrow in width compared to the fetch in front of the dam. The computational formula for wave height, however, is based upon a wide reservoir, i.e. no lateral restrictions. In that case the effective fetch equals the overwater distance. For reservoirs of limited width the effective fetch depends on the shape of the reservoir. A method for determining the effective fetch is presented in Figure 4.5 (Saville et al., 1962).

The effective fetch is determined as the average distance in the direction of the wind within a 90° sector centred about the wind direction:

$$F_e = \frac{\sum_{-45°}^{+45°} R_i \cos^2 \alpha_i}{\sum_{-45°}^{+45°} \cos \alpha_i}$$

where: R_i : radial distance from the shore line to the dam; and
α_i : angle between wind and radial direction.

The shore line at maximum reservoir level is used for determination of the radial distances. The most unfavourable wind direction is assumed to coincide with the orientation of the maximum effective fetch.

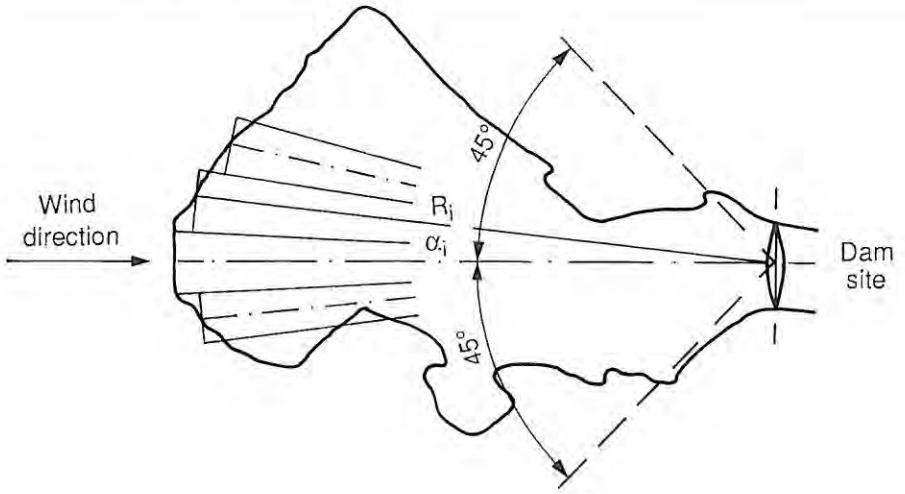


Fig. 4.5
Method for determination of effective fetch

A set of non-dimensional formulae are applied for calculation of wave characteristics, significant wave height, H_s , peak period, T_p , and minimum wind duration, T. For fully developed waves they are:

$$\frac{gH_s}{u_*^2} = 0.0506 \left[\frac{gF_e}{u_*^2} \right]^{\frac{1}{2}}$$

$$\frac{gT_p}{u_*} = 0.903 \left[\frac{gF_e}{u_*^2} \right]^{\frac{1}{3}}$$

$$\frac{gT}{u_*} = 23 \left[\frac{gF_e}{u_*^2} \right]^{\frac{2}{3}}$$

where g : acceleration of gravity; and
u_* : friction velocity.

The friction velocity is calculated from the wind velocity, U_{10} at 10 m height above ground surface by the empirical formula:

$$u_*^2 = C_D U_{10}^2 = (0.0008 + 0.000065 U_{10}) U_{10}^2$$

The formula is only valid for friction and wind velocities given in metre per second.

4.4 Wave Run-Up and Wind Tide

Waves hitting the upstream slope will run up to a height above the mean water level as shown in Figure 4.6.

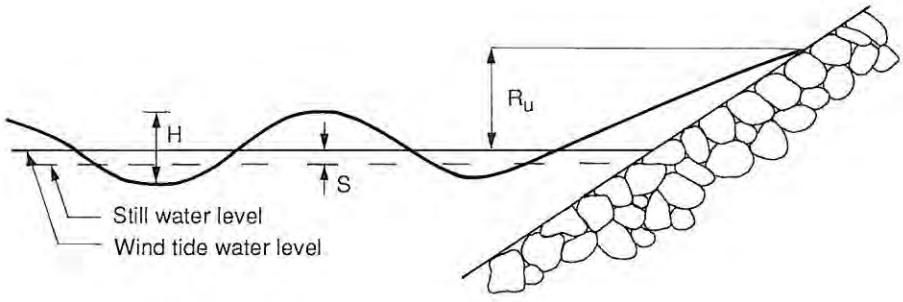


Fig. 4.6
Sketch for wave height, H, wind tide, S, and run-up, R_u, at the upstream slope of an embankment dam

Run-up exceeded only by 2% of the waves in a storm is given by:

$$R_u = rH_s$$

where R_u: run-up, vertical height above still water level;
r: attenuation factor; and
H_s: significant wave height.

The attenuation factor, r, depends on the Iribarren number for the wave slope geometry:

$$I_r = \tan\beta\sqrt{\frac{L_o}{H_s}} = T_p\tan\beta\sqrt{\frac{g}{2\pi H_s}}$$

where I_r: Iribarren number;
ß: slope angle;
L_o: deep water wave length;
T_p: peak wave period; and
g: acceleration of gravity.

The Iribarren number is an expression for the type of wave breaking at the slope:

$I_r < 0.5$	spilling wave
$0.5 < I_r < 2.0$	plunging wave
$2.0 < I_r < 2.6$	plunging or collapsing wave
$2.6 < I_r < 3.1$	collapsing or surging wave
$3.1 < I_r$	collapsing wave

For most embankment dams the Iribarren number will be between 0.5 and 2.0. In Figure 4.7 are given attenuation factors for impermeable and smooth, impermeable and rough, and permeable and rough slopes (Bruun, 1985).

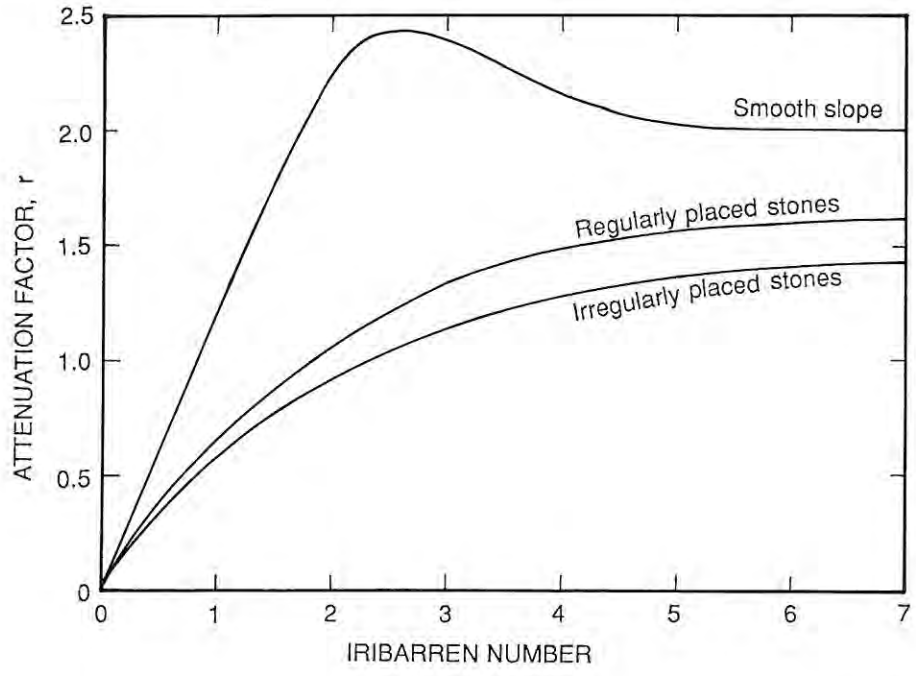


Fig. 4.7
Run-up attenuation factor for typical embankment slopes

Wind tide must be accounted for in shallow reservoirs. The rise of the water due to the shear of persistent wind, the wind tide, S, is estimated from the following equation:

$$S = \frac{U^2 F_e}{4800\, D}$$

where S: wind tide in metres;
U: wind velocity in metres per second;
F_e: fetch in kilometres;
D: average depth of the reservoir along the fetch in metres.

For deep reservoirs the wind tide will be of insignificant size compared to the run-up.

5.
EMBANKMENT DEFORMATIONS AND DISPLACEMENTS

Embankment dams are subject to deformation in the form of compression, extension and shear distortion. Since fill materials in general have no tensile strength, cracking will result from even very small tensile stresses. Even if deformations are not of such magnitude and character as to cause cracking directly, cracking may nevertheless occur by hydraulic fracturing. If the minor principal total stress at any point within the core is less than the pore water pressure at the same level, water pressure may split the core. Splitting (fracturing) may proceed progressively following roughly the plane of the minor principal stress. This occurs if the total stress is low enough and the adjacent material is sufficiently impervious so that water may wedge its way along opening cracks more rapidly than it permeates the pores of the adjacent core material. Sherard (1986) presents a number of examples in an extensive discussion of the phenomenon.

Hydraulic fracturing cannot therefore take place in coarse, pervious materials such as gravel and stones. An impervious core of moraine is, however, susceptible to hydraulic splitting. In dam design it is important that the character and magnitude of the core deformations can be accurately predicted and accommodated. Locations with potentially high shear strains and low principal stresses must be protected by design measures.

Stress and strain conditions can be predicted by means of the finite element method and suitable computer programs. But even the most advanced computer programs imply a simplified version of the real conditions. In particular, the stress-strain properties of the materials involved, as a function of load, environment and time, are difficult to model in a realistic manner for three-dimensional conditions.

Further development of reliable computing methods will largely depend on the acquisition of knowledge of the real dam behaviour through field instrumentation and monitoring. Such measurements will also afford the possibility of assessing the character and magnitude of embankment deformations by developing empirical design rules.

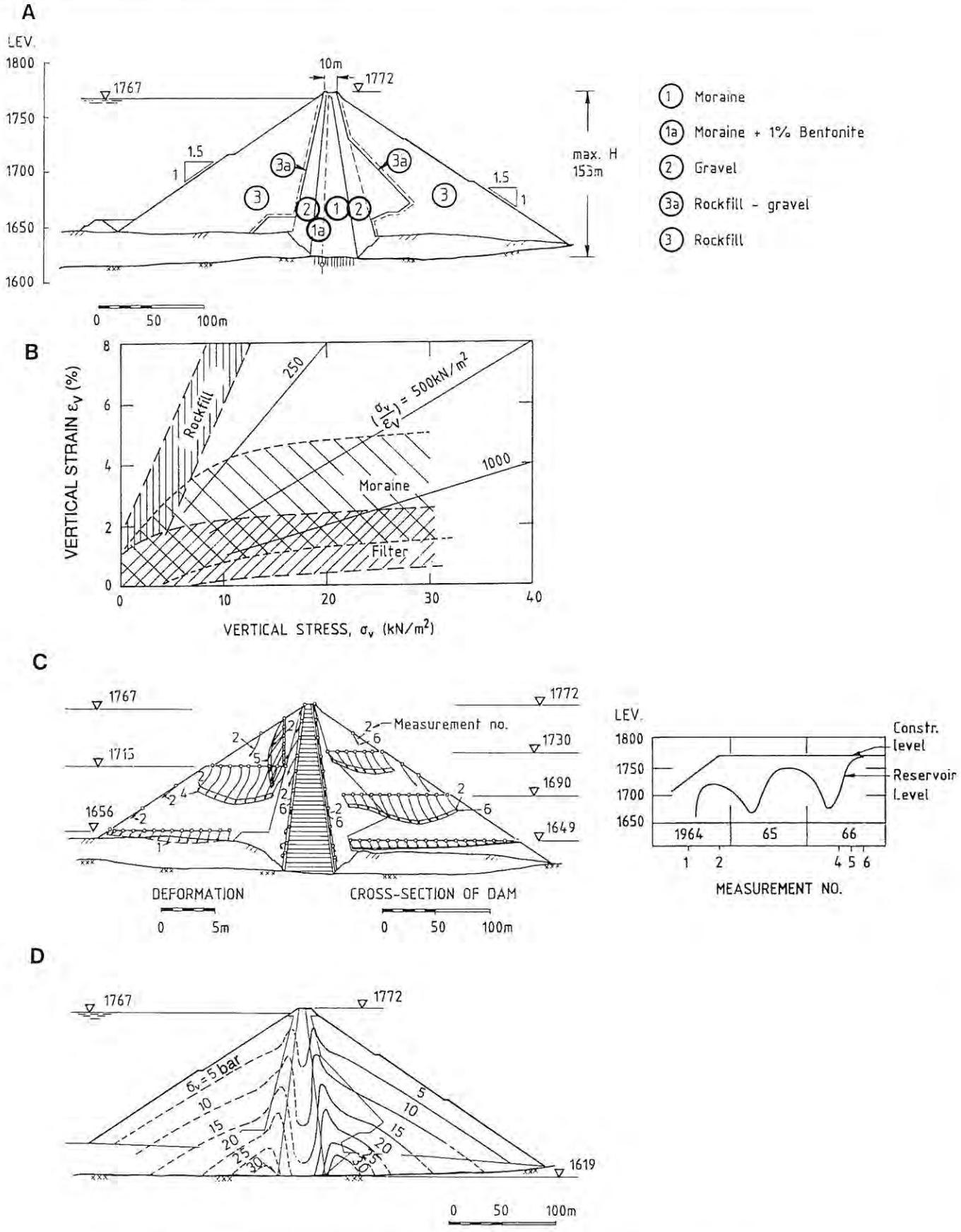


Fig. 5.1 Observations made on the Gepatsch Dam (After Schober, 1967)
A) Cross-section of the dam *B) Compressibility of fill materials*
C) Displacement vectors *D) Vertical stress measured in the cross-section*

Measurements of displacements and stresses have been and are being carried out on a number of dams. Only a few dams, however, have been sufficiently well instrumented and monitored to render a full account of developments within the dam during construction, the simultaneous or subsequent impoundment, and during operation of the dam. The following describes some case studies and results from measurements which have contributed greatly to the understanding and knowledge of the behaviour of embankment dams. The dams are described in three categories according to type of impervious element: earth core, frontal deck, and asphaltic concrete core, respectively.

5.1 Dams with Earth Core

The Gepatsch Dam, Austria, is 153 m high and has a cross-section as shown in Figure 5.1. The dam has steep outer slopes, impervious core of moraine and supporting shoulders of blasted rock. Figure 5.1c shows displacement vectors measured in the supporting rockfill, in the transition zones and in the filters. The displacements shown are accumulated during construction and full impoundment, two years after completion of the dam. The biggest displacements take place at about mid-height of the shoulder rockfills where they amount to as much as 3.5 m. The horizontal components of the displacement vectors on both sides are pointed outward from the central core towards the outer slopes of the dam. The displacements within the supporting rockfills are much bigger than they are within the filter and transition zones.

The results of laboratory oedometer tests, Figure 5.1b, show that the rockfill in the shoulders is more compressible than the moraine in the core, which in turn is more compressible than the gravel in the filter and transition zones.

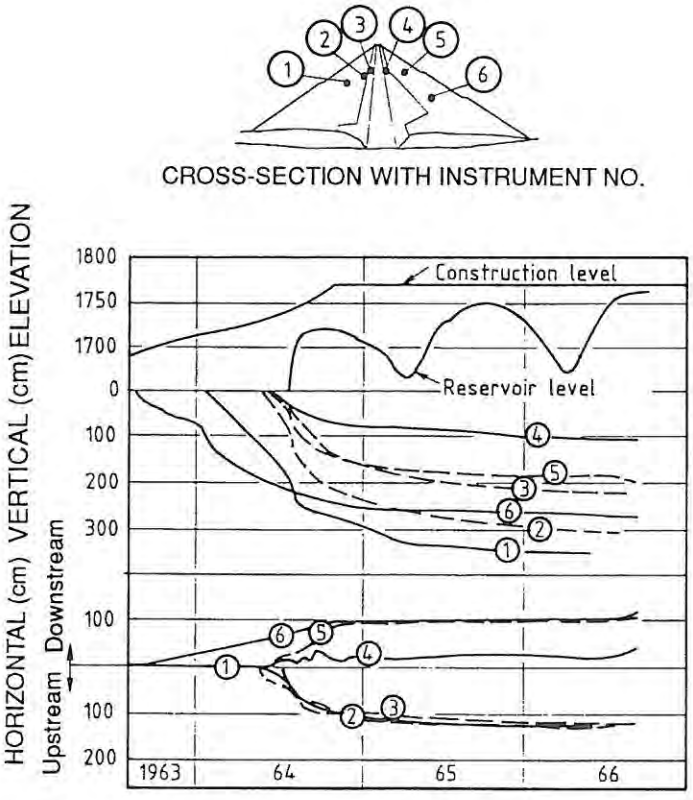


Fig. 5.2
Displacements measured at six points in the Gepatsch dam (After Schober, 1967)

The different compressibilities of the various zones in the dam have brought about comprehensive internal redistribution of stresses compared to the distribution in a homogeneous embankment. Contours through points with equal vertical stresses are shown in Figure 5.1d. Clearly the weight of the core is partially transferred to the adjacent filter zones. The core is "hanging" on the more rigid filter zones. As a consequence of this arching effect the vertical stress on horizontal planes in the central part of the core actually amounts to only about 50% of, γz, at that level (γ = total unit weight, z = depth beneath top of dam).

Figure 5.2 shows displacements accrued during construction and the first complete filling of the reservoir. Two things in particular should be observed: Firstly, that deformations induced during construction are far greater than those caused by the filling of the reservoir and, secondly, that deformations caused by the impoundment are greatest at the upstream side of the core.

El Infernillo Dam, Mexico, is 149 m high and has a cross-section as shown in Figure 5.3. It is a rockfill dam comprehensively instrumented and monitored during construction and the first filling of the reservoir. Figure 5.3 shows deformations measured in the clay core. Vertical and horizontal displacements during construction are schematically illustrated, as well as additional displacements caused by first impoundment.

The greatest displacements, about 2 m, are to be found around mid-height of the dam, as was the case at the Gepatsch Dam. Similarly most of the deformation in the body of the dam was induced during construction. At the top of the dam no measurements can be made before the dam is completed, and thus all deformations here are post construction. The horizontal displacements along the axis of the dam show that the embankment has a tendency to settle towards the deep valley bottom.

Of special interest are the inclinometer measurements which show how displacements normal to the dam axis evolve during the first impoundment of the reservoir. Initially, the core was deflected towards the upstream side as a consequence of the additional settlements in the submerged parts of the supporting rockfill. When the water level was raised further, beyond mid-height of the dam, increasing water pressure counteracted and reversed the upstream movements so that finally, with a full reservoir, the core was pressed downstream of its original position some 0.45 m by February 1968.

It should also be mentioned that the core of the Infernillo Dam, as in Gepatsch Dam, shows a tendency to arch between adjacent filters, entailing significant redistribution of stresses within the dam. For both dams the core material was more compressible than the filter zones.

The Muddy Run Dam, USA (Figure 5.4) gives an example of a 76 m high rockfill dam where the core is less compressible than the rest of the dam embankment. As emerges from the settlement pattern, the supporting shoulders are actually "hanging" on the core. Loads are transferred through

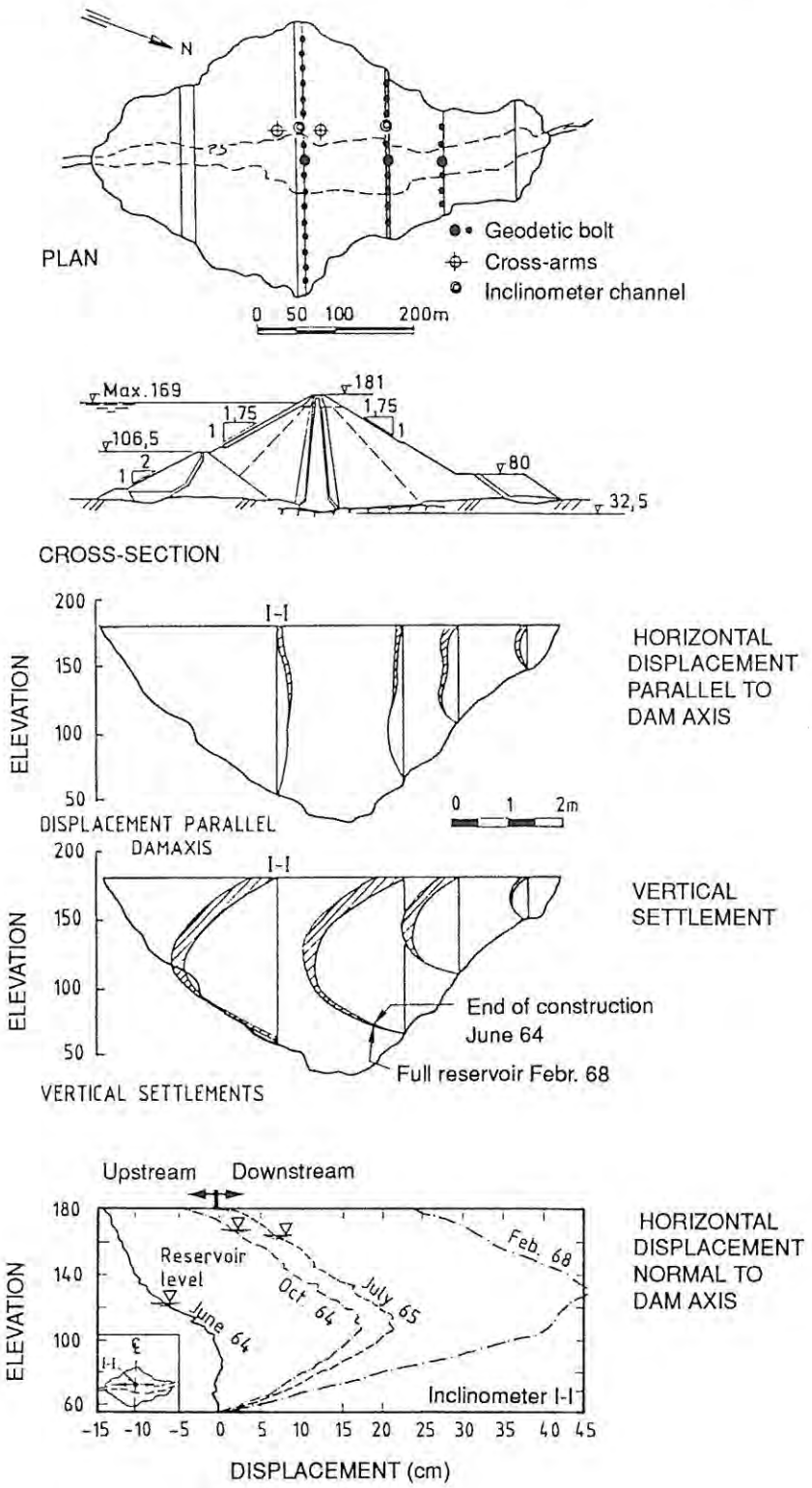


Fig. 5.3
Observations at El Infernillo Dam. Deformations in the core. Vertical settlements are measured on cross-arms, and horizontal displacements are measured by means of inclinometers. (After Wilson and Squier, 1969)

the transition and filter zones so that the vertical stresses in the core are higher than, γz, i.e. total unit weight times vertical depth under top of dam.

Figure 5.4 shows the same pattern of horizontal displacement for Muddy Run as was the case at Gepatsch and El Infernillo. The displacements have an upstream direction on the upstream side and a downstream direction on the downstream side, both during construction and upon impoundment. The core is pressed downstream at full reservoir level.

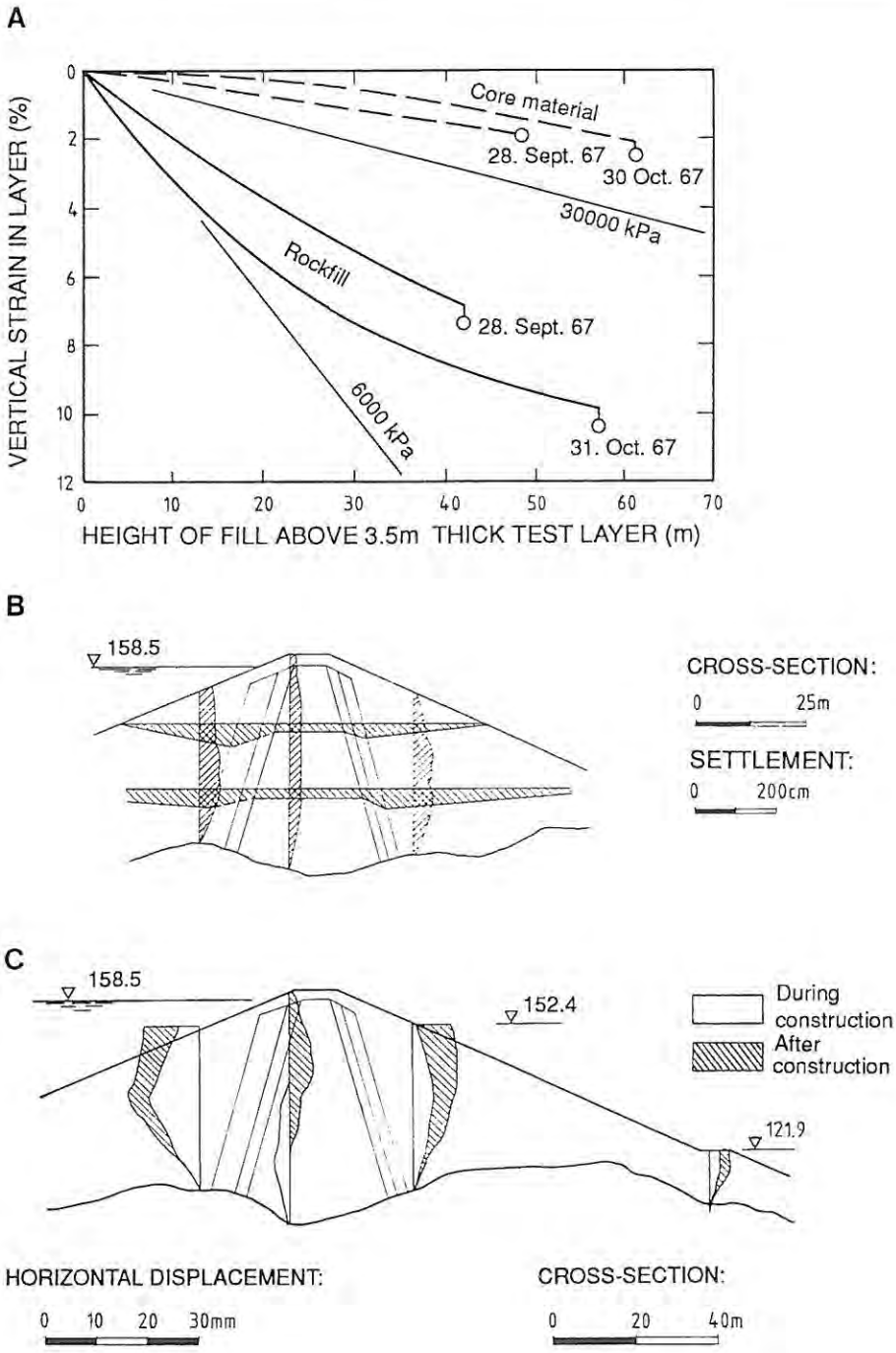


Fig. 5.4
Observations from Muddy Run Dam (After Wilson and Marano, 1968)
A) Compressibility of core and rockfill, respectively
B) Vertical settlements after filling of reservoir
C) Horizontal displacements normal to dam axis

In summary, the dams at Gepatsch, El Infernillo and Muddy Run are all zoned rockfill dams with a central impervious core surrounded by adjacent filter and transition zones. The materials in the various zones exhibit different compressibilities. In the Gepatsch Dam the filter and transition zones are the least compressible parts of the dam, and the deformations show that the core is arching between the adjacent zones. Significant arching of this kind increases the potential for hydraulic fracturing. In Muddy Run Dam the core is less compressible than the rest of the fills whose weight is partly transferred to the stiffer core, thus increasing the core stresses.

For all three dams measurements show that the greatest deformations, and the corresponding redistribution of stresses, take place during the construc-

tion of the dam and not during impoundment. Impoundment is, however, the main cause of the transverse displacement of the core, first causing an upstream movement, then downstream as the reservoir reaches maximum water level.

Longitudinal (axial) displacements generally tend to be directed towards the centre of the valley bottom. Thus, the horizontal compressional stresses increase in the central regions and decrease toward the abutments, where extension and cracking may occur if not properly accounted for.

Svartevann Dam, Norway, is 129 m high and has a total volume of 4.7 mill. m^3 (see front cover photo). At the time of completion (1976) it was the highest dam in Northern Europe. It may be considered typical of Norwegian rockfill dams with moraine core, but the core is narrower than in most dams. The placing and compaction of the rockfill was also somewhat special. For the first time in Norway compaction was carried out without the expense of sluicing, the rockfill being placed in 2 metres lift and compacted by vibratory rollers (Kjærnsli et al., 1982).

The dam is extensively instrumented including measurements of seepage, pore water pressures, earth stresses and displacements. Here, only the deformations will be commented upon.

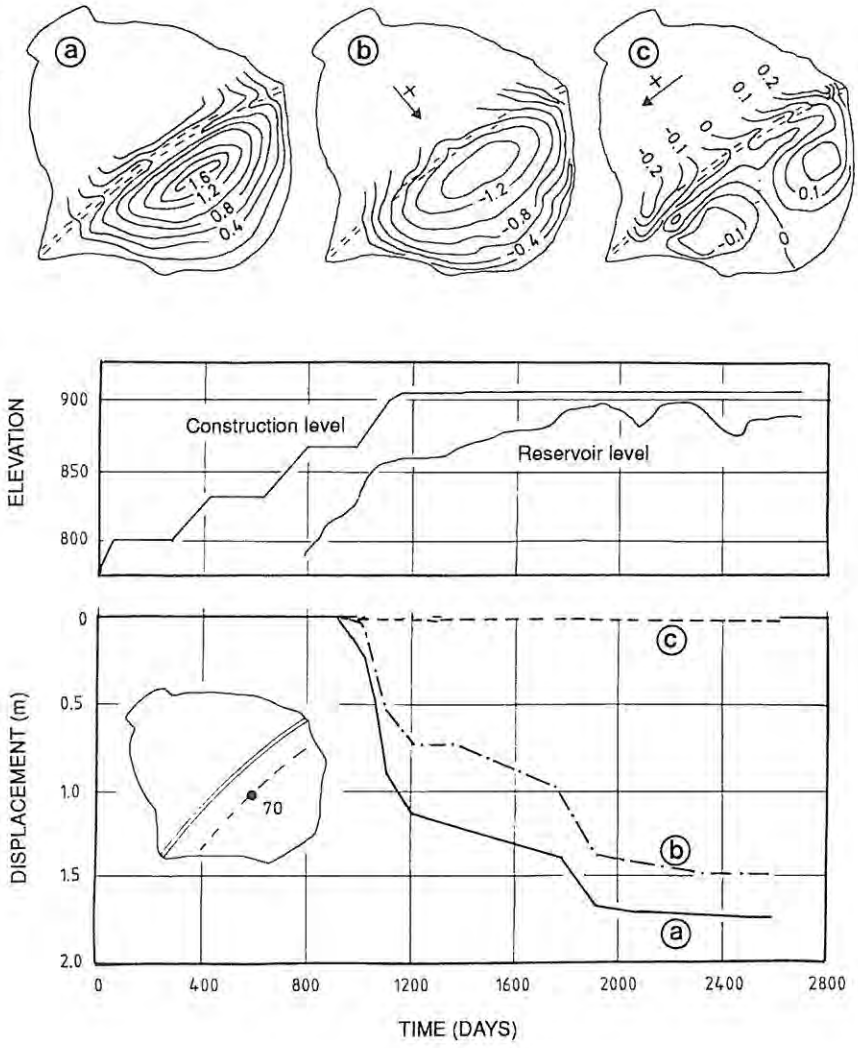


Fig. 5.5
Svartevann Dam – displacements of surface points
A) Vertical settlements
B) Radial displacements
C) Tangential displacements

Displacements were measured at the slope surfaces and within the dam. A total of 160 measuring bolts were installed at the surface of the dam, and their geodetic coordinates were determined as soon as the embankment reached the level of each new row of bolts. Figure 5.5 shows the accumulated surface displacements four years after the completion of the dam. The three components of the displacement vectors, which were determined by geodetic surveying from fixed base stations outside the dam, are drawn as contours through points of equal displacement components. The greatest vertical surface settlements occur at about 2/3 dam height in the downstream slope.

Figure 5.5 also shows displacements over time for bolt no. 70. The vertical settlement accumulated is 1.75 m, and the tangential (axial) displacement is practically nil as the location is close to the plane of symmetry in the dam.

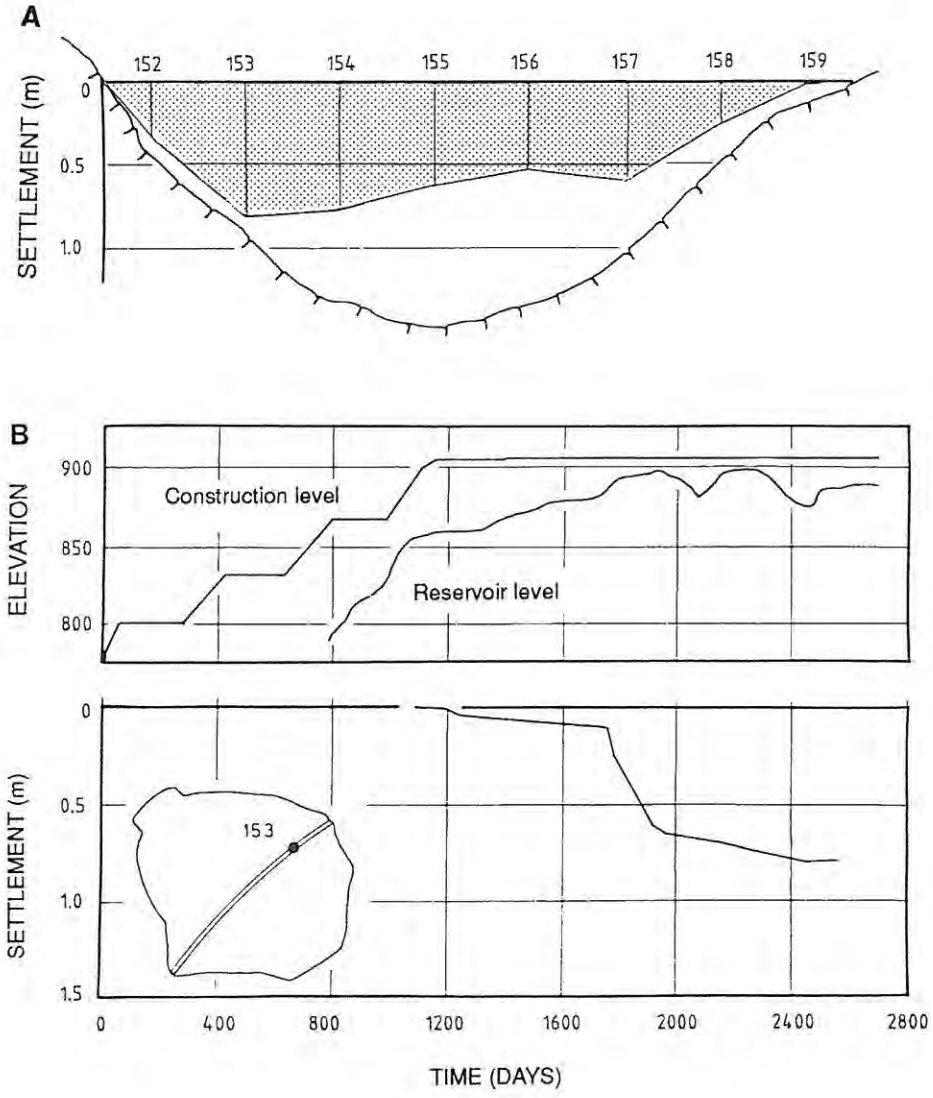


Fig. 5.6
Svartevann Dam – settlements of the top of the core (A) and settlement of bolt 153 with time (B)

The settlements of the top of the core are measured to control the camber and actual freeboard of the core (Section 6.4). Figure 5.6 shows the settlements along the cross valley profile. The greatest settlements occur above the steep valley sides, and bolt no. 153 has settled most, about 0.8 m. The

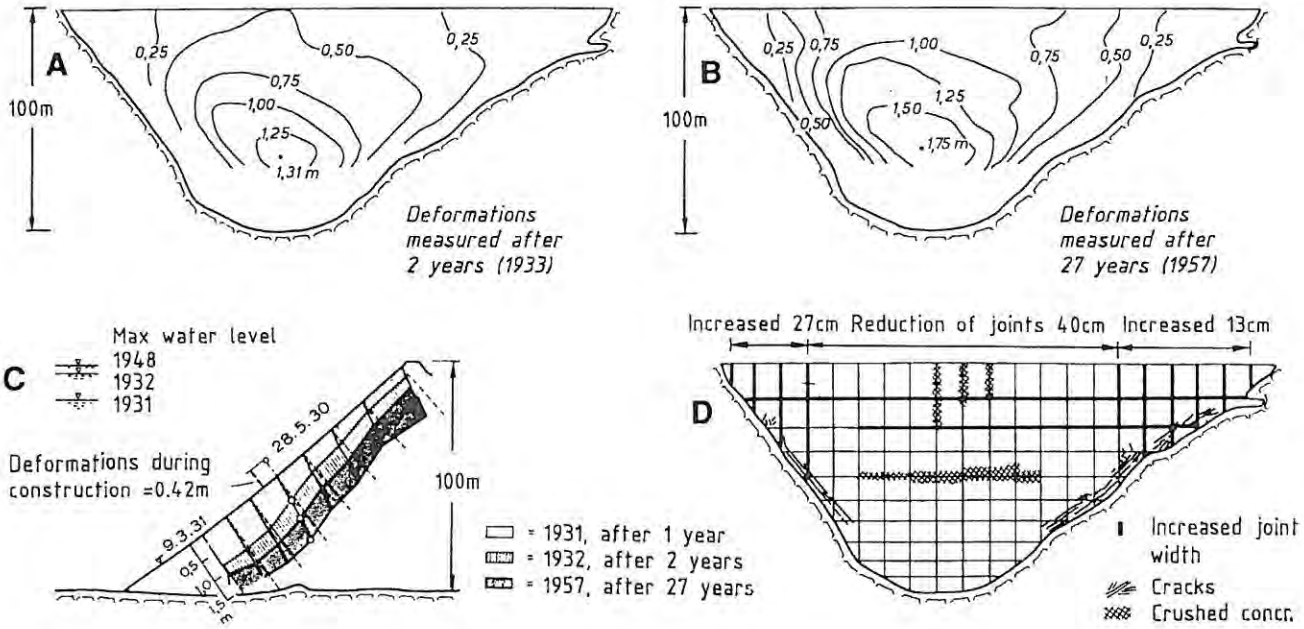


Fig. 5.7 Observations on Salt Springs Dam (After Steele and Cooke, 1958)

A) Displacements normal to the slope of the deck after 2 years of operation

B) Displacements normal to the slope of the deck after 27 years of operation

C) Displacements at the largest cross-section of the dam

D) Measured changes in joint width indicated cracked and crushed concrete in the deck

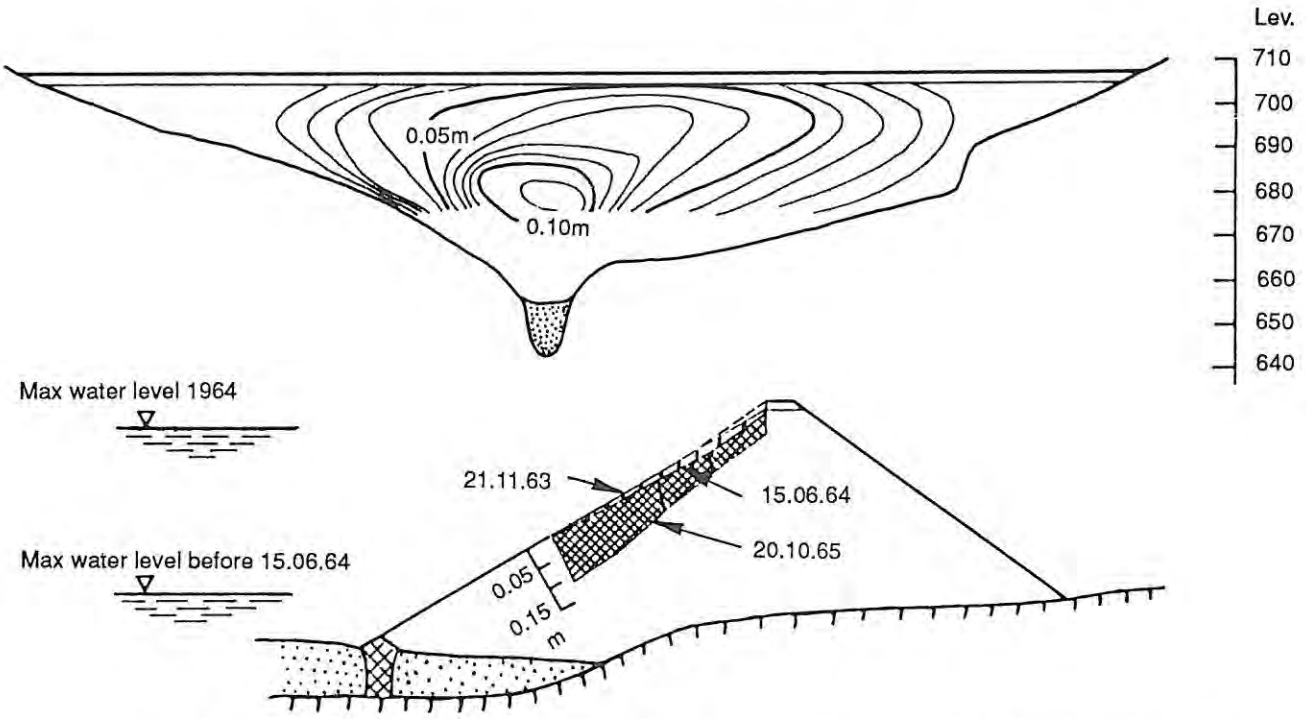


Fig. 5.8 Observations on Venemo Dam. Displacements normal to the slope of the frontal deck

settlement here reaches about 1% of the height of the dam at the bolt location. The figure also gives embankment and reservoir level and corresponding settlements for bolt no. 153 with time. About 80% of the total settlement took place within a comparatively short period of time during the first filling of the reservoir.

Measurement of displacements within the dam are limited to the central cross-section and both abutments. Figure 1.8 shows displacements of inclinometer casings in the main cross-section four years after the dam was completed. The largest measured displacements have taken place in the downstream shoulder where the displacement vector totals about 3 m four years after completion. The radial displacement of the core is about 1.3 m towards the downstream side.

5.2 Dams with Frontal Deck

Salt Springs Dam, USA, is a 100 m high rockfill dam. It represents a much earlier construction technique than any of the dams described above. The rockfill was dumped in lifts of up to 50 m height and sluiced with water. The rockfill embankment was completed in December, 1930, and the concrete deck was cast during the second half of the same year. The filling of the reservoir started in March 1931.

Measurements (Figure 5.7) show that the concrete deck has been subjected to relatively large deformations. At about 1/3 dam height the displacement normal to the slope measured 1.75 m after 27 years. Measurements of joint gaps show decreasing width in the central part of the dam where local crushing of the concrete has occurred, but towards the abutments the joint gaps are increased. These observations clearly demonstrate that the rockfill settles towards the deep valley bottom.

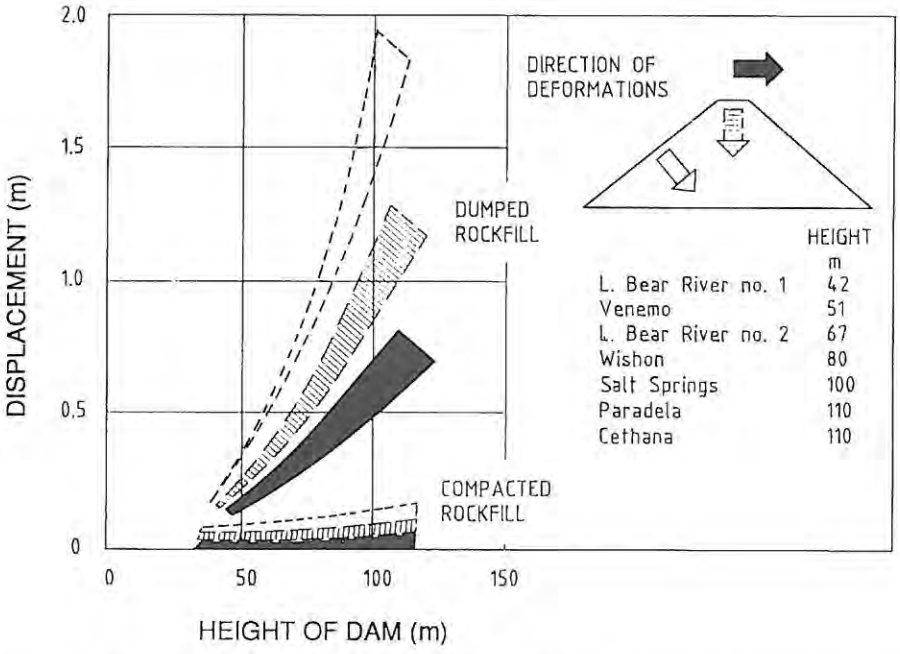


Fig. 5.9
Measured displacements after embankment completion from dams consisting of dumped rockfill as opposed to layered, compacted fill, all with frontal deck. Venemo and Cethana dams represent layered, compacted fill.

Venemo Dam, Norway, is a 64 m high rockfill dam with an asphaltic concrete deck (Figure 5.8). The fill consists of tunnel spoil placed in horizontal,

1.5-2.0 m thick layers which were sluiced with water and compacted with a vibratory roller. The biggest displacement normal to the slope of the deck is only 0.11 m, which is very much smaller than the corresponding value at Salt Springs. Apart from the difference in height, the essential difference between the dams is the way of building the fills. In Salt Springs fills were dumped in high lifts and sluiced, whereas the Venemo Dam fills were placed in 1.5 to 2.0 m thick horizontal layers which were sluiced and compacted with vibratory rollers (Kjærnsli and Torblaa, 1966).

The importance of compaction becomes evident from Figure 5.9 which presents a survey of results from a number of rockfill dams with frontal, impervious deck. The deformations of compacted layered rockfill are only a small fraction of the deformations of loose, dumped rockfill (Soydemir and Kjærnsli, 1979).

5.3 Dams with Asphaltic Concrete Core

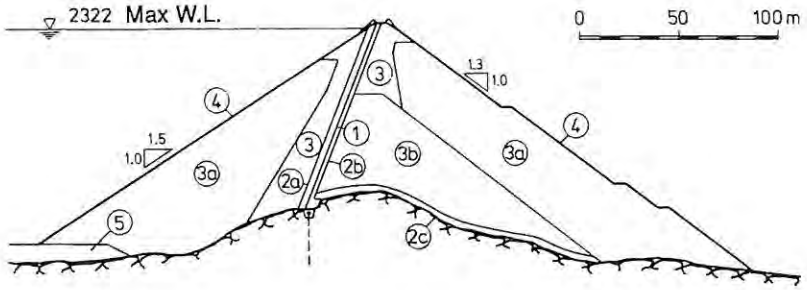
Finstertal Dam, Austria, completed in 1980, is a 150 m high rockfill dam with a 96 m high inclined asphaltic concrete core resting on a rock foundation ridge (Pircher and Schwab, 1988). The dam is exceptionally well instrumented and monitored, but here only the maximum displacements in the sluiced and heavily compacted rockfill will be reported. At the end of construction the maximum settlement in the rockfill at about mid-height was only 0.3 m. It increased to 0.35 m at full reservoir level and was 0.40 m after the first complete drawdown 3 years later. The maximum horizontal displacement at the end of construction was only 0.10 m increasing to 0.17 m at full reservoir. These very small displacements demonstrate the extremely beneficial effects of compacting high quality rockfill in relatively thin layers. Compared to the results from Gepatsch Dam, the displacements in Finstertal are one order of magnitude smaller.

Storvatn Dam, Norway, completed in 1987, is 90 m high and has a total volume of approximately 10 mill m^3. It is one of seven large Norwegian rockfill dams with a central core of asphaltic concrete (see Table 7.1). A cross-section of the dam is shown on Figure 5.11. In addition to the information given on this figure regarding layer thickness, it should be noted that all zones placed in layers were compacted with vibratory rollers, the zone 4a also being sluiced with water (for definition of zones, see Figure 1.3).

Figure 5.11 shows results of displacements of inclinometer casings in the main cross-section. At end of construction in October 1986, when the water level was 15 m below full reservoir, the largest measured displacements were about mid-height in the downstream supporting fill, zone 4b, where the vertical settlement was 0.35 m and the horizontal displacement 0.12 m. Three years later, at full reservoir, the corresponding displacements at the same point were approximately 0.5 m and 0.2 m.

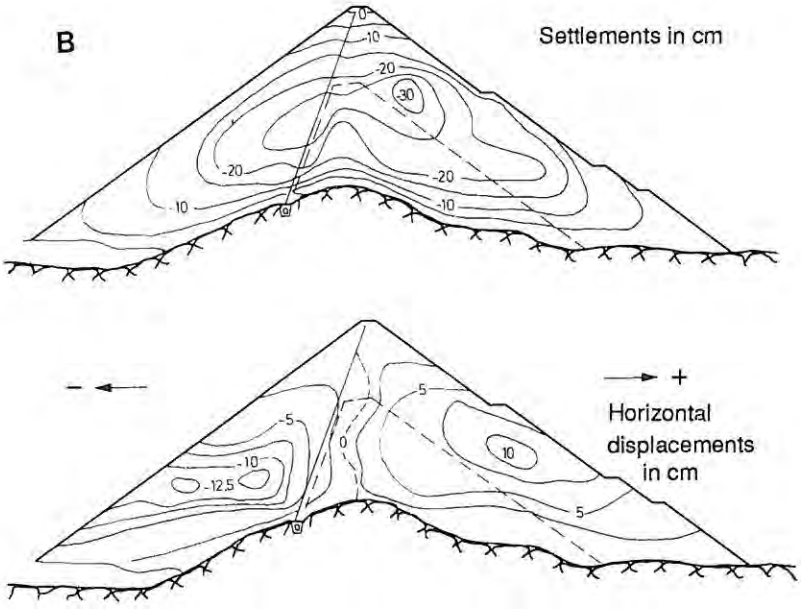
The shapes of the deflection curves of the horizontal tubes at elevations 986 and 1016 clearly show that the central part of the rockfill is much less compressible than the outer part. The central part, zone 4a, was placed in

A

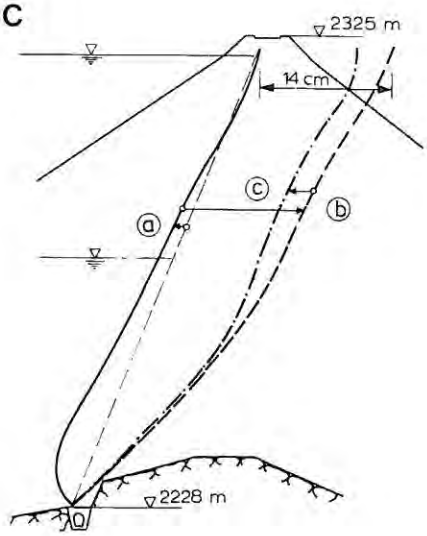


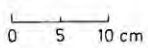
	Zone	Material	d_{max} [mm]
1	Impervious core wall	asphaltic concrete	16
2a	Transition zone upstream	screened-out moraine	100
2b	Transition zone downstream	screened-out quarry material	100
2c	Drainage zone	quarry material	700
3	Shoulder	quarry material	700
3a	Shoulder	quarry material	700
3b	Shoulder	moraine	700
4	Surface course	blocks	500–1000 (length of stone)
5	Remaining overburden	moraine	

B



C





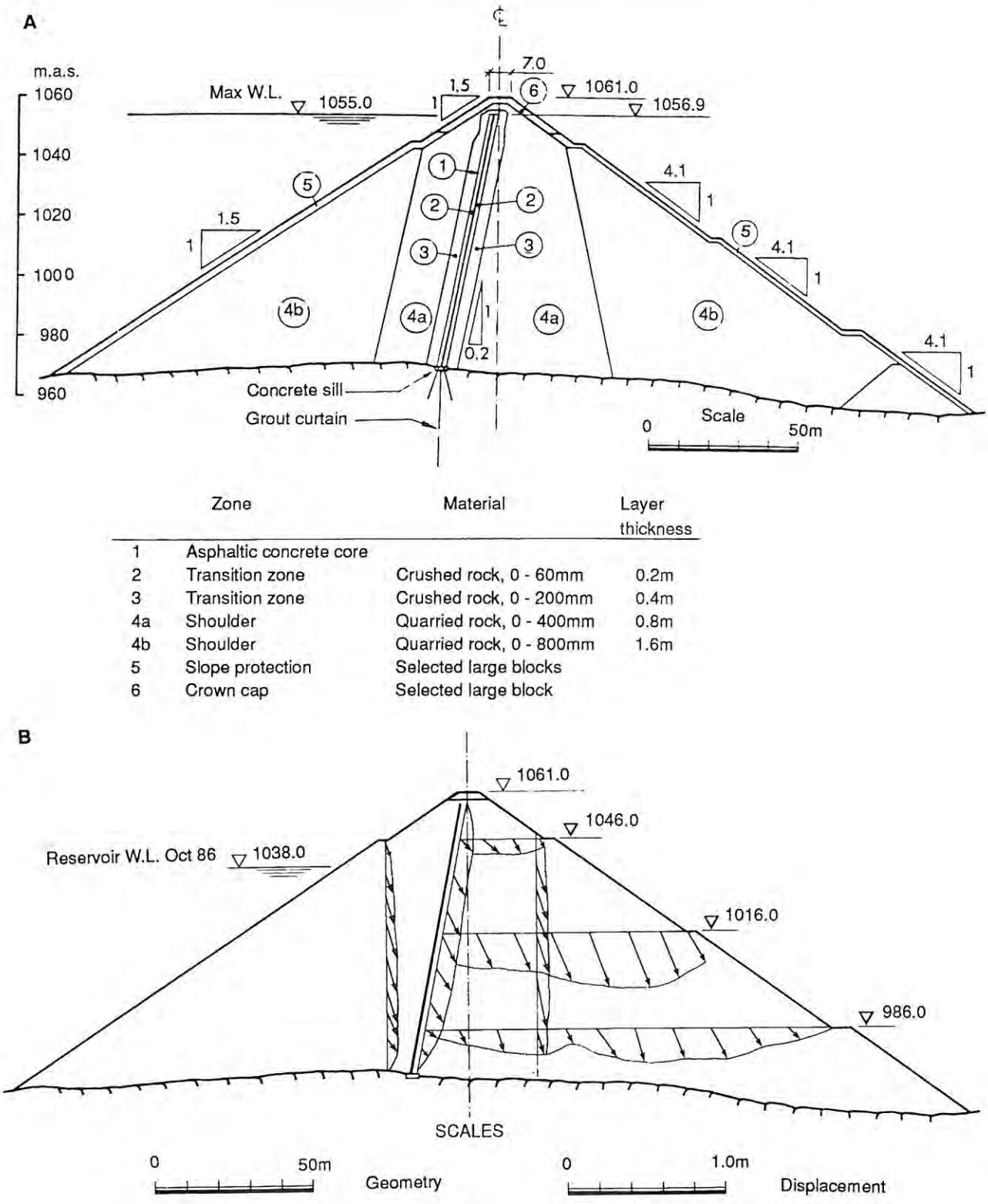
a Deviation from the position as planned at end of construction
b First impounding to max. storage level
c Following drawdown

Fig. 5.10 Finstertal Dam – measured vertical and horizontal displacements (After Pircher and Schwab, 1988)

A) Cross-section

B) Displacements at end of construction

C) Horizontal displacement of core after first impounding and after subsequent drawdown



Zone		Material	Layer thickness
1	Asphaltic concrete core		
2	Transition zone	Crushed rock, 0 - 60mm	0.2m
3	Transition zone	Crushed rock, 0 - 200mm	0.4m
4a	Shoulder	Quarried rock, 0 - 400mm	0.8m
4b	Shoulder	Quarried rock, 0 - 800mm	1.6m
5	Slope protection	Selected large blocks	
6	Crown cap	Selected large block	

Fig. 5.11 Storvatn Dam – measured vertical and horizontal displacments

A) Cross-section

B) Displacements of inclinometer casing at end of construction with simultaneous filling of reservoir (80% full)

layers of 0.8 m, sluiced and vibrated whereas the outer part, 4b, was placed in layers of 1.6 m and vibrated with no sluicing. If no extra effort had been put into zone 4a in relation to 4b, the settlement of the core wall might well have been twice as much as what was measured (Arnevik et al., 1988).

The displacements measured at Storvatn Dam should be compared to the corresponding displacements measured at Svartevann Dam. The large difference cannot be attributed to the difference in height alone, i.e. 90 m and 129 m. The measurements at these two dams again show how much the compressibility of compacted rockfill depends on the thickness of compacted layers (0.8 m and 1.6 m at Storvatn and 2.0 m at Svartevann) and the effect of sluicing.

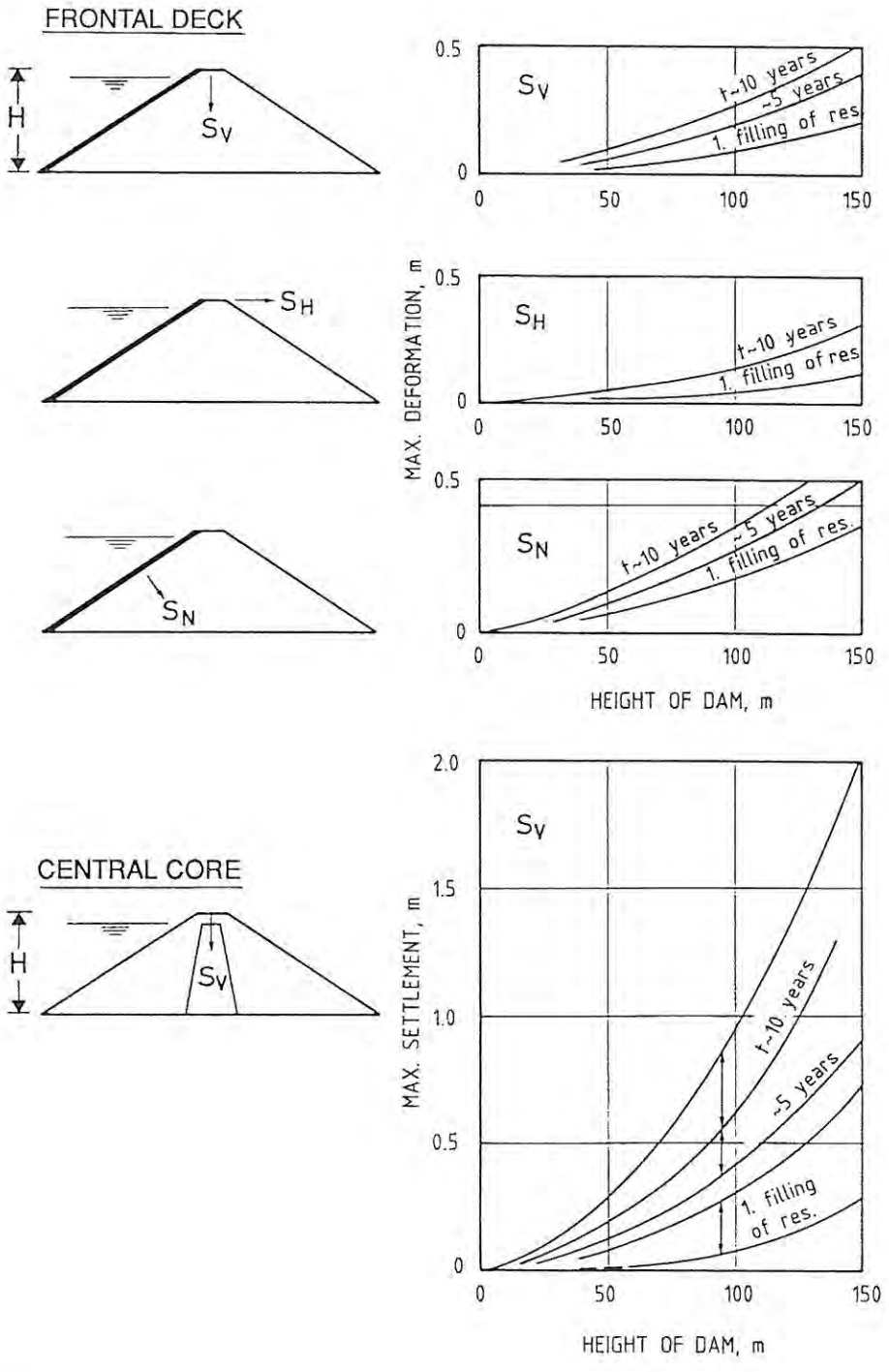


Fig. 5.12
Settlement as an empirical function of dam height and time for completed dams with layered, compacted rockfill.

5.4 Post-Construction Deformations

The displacement curves on Figure 5.12 are based on observations made over long periods of time for a number of rockfill dams. The displacements increase consistently, but at varying rates according to the age and height of the dam. The magnitude of the settlement clearly depends on the degree of compaction achieved in the rockfill. Empirically it has been found that the creep displacements accumulate approximately linearly with the logarithm of time.

For dumped and loosely packed rockfill, the settlements follow the same basic pattern of time and height dependency, their magnitude, however, are 5-10 times bigger than for a layered and compacted rockfill of similar age and height. Clements (1984) and Dascal (1987) present results from a number of dams.

If the water level in the reservoir is raised more or less in step with the embankment fill, then a major part of the settlements caused by submersion will take place during construction. Post-construction settlements will be very much smaller than if the reservoir is filled after construction is completed. Another advantage with the slow rate of reservoir filling in step with the embankment construction, is that the dam structure and the impervious element have time to gradually adjust to the changes in load. This reduces the likelihood of cracking and hydraulic fracturing (see Section 1.5).

6.
DESIGN AND ANALYSES

Design of rockfill dams must be adapted to geological and topographical conditions at the site. Under any given circumstances, however, alternative solutions will be feasible, and other factors may determine the final choice.

6.1 Choice of Impervious Element

The type of impervious element, that is, the selection of material and its position in the dam, is the first and foremost consideration in the planning of an embankment dam (Kramer, 1988).

When soil of suitable quality is available within an economic distance from the site, the dam should generally be designed with an impervious core of soil. The width of the core has to be decided. Soil cores with widths of half the water head or more at that elevalation, are considered "wide", whereas cores of widths of one third of the water head or less, are considered "narrow". In general, a wide core is considered to be safer than a narrow one. The volume of available material, or their technical properties such as rate of consolidation, may, however, restrict the width of the core.

A wide core may be given a central, symmetrical and vertical position in the dam. A narrow core should preferably be given an inclined position sloping towards the upstream side. The inclined position produces safer stress conditions and thus reduces the risk of arching and cracking in the core. However, a sloping core of low shear strength may create stability problems for the upstream embankment slope. The width and position of the core must be chosen with due consideration to the topography of the site. If the dam is set in a narrow gorge with steep valley slopes, the core should be wide and vertical.

In economic terms moraine cores have proved preferable to other types in most Norwegian dams. However, where moraine is not available, or is in short supply, other impervious materials have to be used. Clay, silty materials and crushed stone ground to sufficient fineness may be used for impervious cores following the same guidelines as for moraine (Chapter 3).

Concrete may be used for waterproofing. In low dams it may be placed in a central vertical wall. The wall should be comparatively thin, approximately

30 cm, reinforced on both sides and appropriately jointed to reduce cracking, both during settling and curing, and during the subsequent placing of fills and later impoundment. Schober (1988) reports the construction of a central concrete core with a bitumen layer on either side to reduce the interface friction and stresses in the core.

For dams of more than 10 to 15 m height a frontal concrete deck is considered preferable to a central concrete wall. A frontal deck should be at least 30 cm thick. Dual side reinforcements, and vertical and perimeter joints must be used to allow for the anticipated movements of the embankment with respect to the fixed concrete plinth boundary of the concrete deck. For dams of this type, reference is made to the publications by Cook and Sherard (1985,1987).

Bituminous impervious elements are well suited to embankment dams. Compacted asphaltic concrete may be applied in a central core wall or in a frontal deck (Arnevik et al., 1988; ICOLD, 1992). Existing central walls have a maximum thickness of about 1% of the water head and minimum thickness 50 cm. A frontal deck should consist of at least two layers each of 6-7 cm thickness.

A central wall of pebble size stones saturated with bitumen may serve as impervious element. The voids of the uniformly graded stone material are filled with hot bitumen. Adjacent filters must, therefore, be impermeable to bitumen under the actual temperature conditions, i.e. the filters must have a sufficient fines content to prevent any flow of bitumen from the wall.

In special cases sealing by means of wooden planking have been used. A wall or deck of wooden planks should consist of two layers of impregnated groove and tongue planed planks of at least three inch thickness. Sometimes an impervious sheet material is placed between wooden plank layers.

Single or multi-layered membranes of synthetic plastic or rubber sheets welded together may serve as impervious elements in minor dams. So far such synthetic membranes have not been extensively used, but geosynthetics in general are rapidly gaining ground in geotechnical engineering.

When the dam is founded on rock, any of the above mentioned sealing elements may serve the purpose. If, however, the dam is to be founded on overburden deposits of considerable thickness, an earth core would be preferable, as it will tolerate and adjust to more severe shear distortions than any other alternative.

6.2 Layout and Foundation Preparation

The foundation of a dam is characterized by topography, load bearing capacity, compressibility and permeability. Design of the embankment and measures to control seepage and pore water pressures in the foundation must take these characteristics into account; they must suit the ground conditions. Dam foundations for practical purposes, fall into two main categories: Solid rock or loose ground. Then there are a number of conditions in-between.

In Norway, rock foundations are generally of sufficient strength and rigidity for any type of dam. With the exception of rare cases where water soluble limestone has led to cavities (karst), rock foundations can be made sufficiently watertight by regular grouting. As a rule a grout curtain is extended down to a depth of not less than one third of the height of the dam, or to a minimum of 6-10 m below the rock surface.

If the permeability of the rock at these depths is still too large, grouting is automatically carried on until satisfactory rock is encountered; or to a maximum depth of two thirds of the height of the dam. It should be observed that it is difficult to establish a sufficiently impervious contact between the grout curtain and the core when the core base is narrow. A wide core is therefore preferable if the foundation rock is of poor quality.

If the surface rock is of exceptionally poor quality, weathered or affected by permafrost, it may have to be removed by excavating a trench down to unaffected or unweathered rock. A core foundation trench must have sloping sides (for example 1:1) and be sufficiently wide (4-5 m at the bottom) to prevent arching across the trench and the ensuing loss of vertical soil stress in the core. Low total stresses in the trench may lead to hydraulic fracturing at this location (Chapter 5). The bottom and sides of the trench must be prepared and treated as required for the core foundation.

At dam sites where the bedrock is covered by overburden to great depths, the bearing capacity, compressibility and permeability of the overburden material will be of decisive importance for the project. In these circumstances, dams with impervious cores of moraine, clay or silty soil are to be preferred. If the overburden consists of material of low bearing capacity such as clay, an earthfill dam may be the best choice. Such ground would require gentle embankment slopes and no advantage can be taken of the high shear strength of rockfill. Steeply sloping rockfill dams are therefore better suited to high-strength foundations such as rock, consolidated moraine or gravel.

In Norway, at altitudes where large dams are sited, the overburden normally consists of materials other than clay, and mostly of moraine or gravelly fluvial deposits. Whenever it is technically feasible and economically acceptable, the core should be extended down to bedrock by excavating a wide trench with gently sloping sides. The load bearing and draining capacities, as well as the compressibility of the unexcavated overburden are important for the stability and deformations of the dam.

If the depth of overburden is so great as to make trenching down to bedrock uneconomical, the sealing of the overburden then becomes of major concern in the project. Figure 1.6 in Chapter 1 gives examples of Norwegian rockfill dams with and without foundation trenches down to bedrock. With the exception of the Mänika and Nerskogen Dams, all dams without a trench are founded on sufficiently impermeable moraine. At Mänika and Nerskogen Dams, adequate sealing was achieved by extending the core with an impervious blanket covering the ground into the reservoir up-

stream. Stability has been secured by a ballasting filter blanket extending downstream from the embankment toe.

If the overburden consists of thick deposits of gravel or coarse sand, sealing by means of grouting must be considered. Grout curtains have been successfully constructed to depths greater than 100 m under high dams. For intermediate depths, where open trenching is not feasible or economical, cut-off walls have been established in permeable deposits by the use of the slurry trench method. The deep, vertical slit trench is during excavation filled with a stabilizing slurry which is subsequently displaced by a permanent cut-off wall material such as concrete or clay.

For dams founded on soil overburden it is essential to secure free drainage of any water from seepage and leaks through the foundation. Seepage water must not be allowed to set up high pore pressures, which could threaten the stability of the dam or cause erosion in loose materials underneath the dam. To meet both requirements, the drainage system, usually combining filters, wells and ditches as necessary, should be designed to trap the water immediately downstream of the cut-off wall. This will secure a free exit and safe outlets for the water collected. Additionally, a ballast filter downstream of the dam may be appropriate. The need for such measures is analysed by means of finite element computer models or flow nets drawn by hand (Chapter 4). To monitor the actual drainage conditions, pore pressures in the foundation are measured by means of piezometers or standpipes as presented in Chapter 8.

Concrete structures in conjunction with embankments should preferably be founded on rock. Concrete surfaces abutting the earth core must be free of protruding ledges and should be inclined away from the core to minimize the risk of cracks along the soil-concrete interface. Inclinations in the order 10:1 have proved satisfactory.

Special attention should be paid to interior culverts which have been the cause of failures of several dams internationally. Culverts should preferably be cast in rock trenches with the top more or less on level with the surrounding rock surface. Culverts may also be set on the rock surface, but the part of the culvert protruding into the core should have sloping sides, for instance 1:1, to avoid a local zone of low soil stresses susceptible to hydraulic splitting in the core.

The alignment of the dam is chosen with a view to obtaining minimum dam volume. The dam axis should preferably be convex towards the upstream side. The topography of the site may, however, be such that an axis convex towards the downstream side would entail significantly less volume. The savings from a volume reduction have to be compared with the cost of measures required to cope with the increased risk of cracking in the dam (Arnevik et al., 1988).

The likelihood of unfavourable deformations and possible cracking in the core is analysed from the topography of the site by studying maps supple-

mented with cross and longitudinal profiles of special interest. The cross-valley profile should be smooth without precipices or abruptly changing slopes. Overhanging or steep rock walls are to be avoided or flattened by blasting. Local narrow gorges and deep clefts must be backfilled with concrete (Chapter 7).

Balancing the core along a high ridge with the supporting shoulder fill sitting on rock slopes dipping to one side or both, may create longitudinal cracks in the core. The horizontal stresses in the core are also reduced, rendering the core susceptible to transverse cracking by hydraulic fracturing.

6.3 Need for Foundation Gallery?

In many countries it is common practice to build a gallery under the impervious core. The purpose is inspection and drainage control and to facilitate grouting, if required, of the foundation and possibly of the core after the embankment is completed. The gallery may consist of a concrete culvert built in a trench under and along the dam axis or a tunnel in rock at some greater depth in the foundation. None of the Norwegian large embankment dams have such a gallery. The cost-benefit effect has in general not been found favourable, and the gallery has not been deemed necessary or desirable from a safety point of view. The 90 m high Storvatn Dam with an asphaltic concrete core will be used as a case study in the following discussion (see Figure 5.11).

It was decided that the asphaltic core should be founded on a concrete structure in a shallow rock trench. Furthermore, this structure should be anchored into the rock foundation, and rock grouting should be performed. A main question was, however, whether this concrete structure should be a simple concrete sill (plinth) or a concrete gallery.

The extra cost of erecting a gallery was estimated to ca. 10% of the total cost of the dam, and erecting a gallery would possibly require a prolongation of the time of construction by one year. Would the advantages of a gallery be worth the extra construction and time cost?

The advantage of carrying out the primary construction grouting from a gallery was looked upon as minor or nil. The advantage of a grouting gallery would therefore depend on the need for supplementary (secondary) grouting during the lifetime of the dam. Based on the results from ten exploratory diamond drilled holes to a depth of more than 50 m, the likelihood that the foundation grout work would need future repair, was looked upon as very little. In any case a possible future increase in permeability of the rock foundation was evaluated to be of little economic significance and could be reduced by grouting from tunnels driven under the dam, if really required. If leaks should develop through the asphaltic concrete core, it would not be any easier to grout from a gallery than from the top of the dam. A gallery under the core was not required for the instrumentation and collection system to measure and localize potential leakage in a reliable manner. Thus, in the final design, the decision was made not to include a gallery.

However, for the planned 120 m high Storglomvatn Dam (Table7.1), a gallery is incorporated. The dam rests partly on a moderately karstic rock foundation, and it is very difficult to predict the amount of seepage under the dam. Internationally, there are several reports of dams on karstic foundations where leakage problems have developed. For the Storglomvatn Dam a tunnel has been designed at 25-40 m depth in the karstic rock region under the core base to facilitate grouting, if required.

6.4 Seepage Estimates

The interpretation and use of flow nets was briefly presented in Chapter 4. From a flow net with a mesh consisting of "squares", the quantity of seepage through a 1-metre long section of the dam is given by the equation (Figure 4.1):

$$q = \frac{n_f}{n_d} \, kH$$

where n_f : number of flow channels;
n_d : number of equipotential drops;
k : permeability; and
H : total potential drop from upstream to downstream

The ratio, n_f/n_d , represents a geometrical form factor. This ratio is determined fairly accurately from even very approximate flow nets. The main uncertainty in the computation stems from the determination of the permeability, k, for which estimates may easily vary by a factor of 10. Moreover it may be much greater in the horizontal than in the vertical direction depending on the character of the core material and the way it is placed and compacted. Flow nets which account for anisotropic permeability and flow across interfaces between different zones, are presented by Cedergren (1989).

In order to compute total seepage through the dam, it must be divided longitudinally into representative sections and flow nets drawn for each section. The total seepage is calculated by summing the seepage for all sections (Figure 6.1).

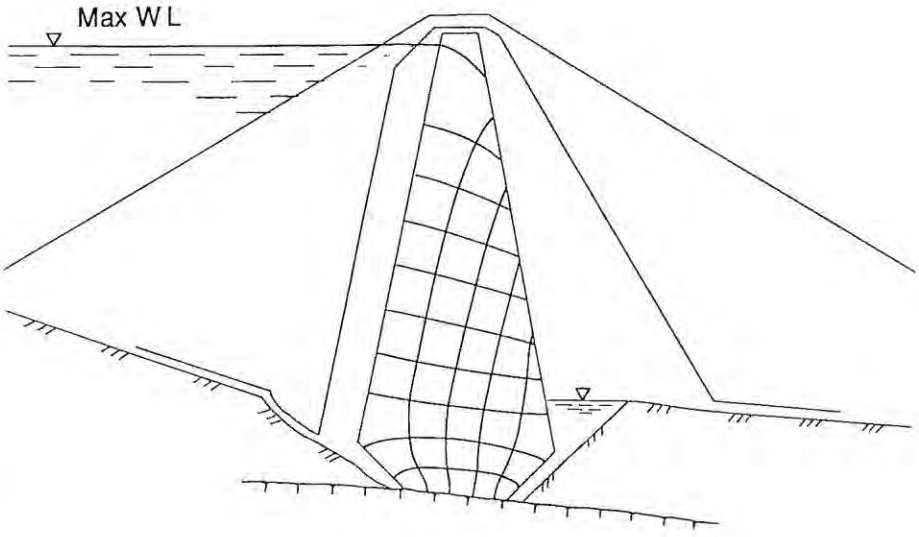


Fig. 6.1
Flow net for a moraine core in a rockfill dam

6.5 Freeboard and Camber

The top of the dam should have sufficient freeboard to prevent waves from splashing over the dam at highest flood water level. The required minimum freeboard is determined as the sum of the maxima for flood water rise, wind tide, wave run-up and possible seiche effect (Chapter 4).

The core should have sufficient freeboard to prevent overtopping. As the effects of waves and run-up are attenuated before the water reaches the core, the freeboard of the central core relative to the design flood water level is commonly stipulated equal to 0.5 metres. Similarly, the freeboard to the top of the dam relative to the design flood water level has been set equal to 2.5 metres as a minimum.

For dams with frontal deck, the deck must have a freeboard of sufficient height to prevent waves from washing over it. Often the deck is topped off with a vertical wave-breaking wall, mainly to save costs. The freeboard required in such cases is set with reference to the top of the wave breaker and may have to be checked by model tests.

Several failures of minor embankment dams in Norway and abroad have been due to insufficient freeboards. The main cause has frequently been an underestimate of the rise of water levels during high floods. Freeboard requirements, therefore, should be estimated with ample safety margins, which will depend on the reliability of the hydrological statistics available and the type of dam involved. Earth fill dams require higher freeboards against flowing water and overtopping than do the less vulnerable rockfill dams. Higher freeboard improves safety against overflow damage, and will also augment compressional stresses and thus reduce the risk of cracks in the top of the core.

Settlements in embankment dams continue after the dam is completed. To ensure that required freeboard levels remain unaffected even after many years of operation, a dam is built with a sufficient camber to compensate for future settlement. A camber, expressed as a fixed percentage of the dam height, is appropriate for dams with a fairly smooth cross-valley profile without steeply inclined abutments. The relative settlement may, however, vary considerably over a cross-valley profile with steep or abruptly changing inclinations as shown in Figure 5.6. Relatively large settlements are likely to take place above the steepest part of the profile. In such cases the camber should be stipulated in fixed dimensions over the cross-valley profile, rather than in percentages (Figure 6.2).

The camber should cover expected settlements allowing a certain margin of safety. For well compacted fills of good quality rock, founded on rock, a camber of about 0.5-1% would be satisfactory. For fills not compacted or of lower quality rock, the camber should be 1.5-2%. The top of the core is given the same camber as the top of the dam.

The camber must be considered so that the outer slopes are kept to the designed inclination and coincide with the designed width of the top of dam

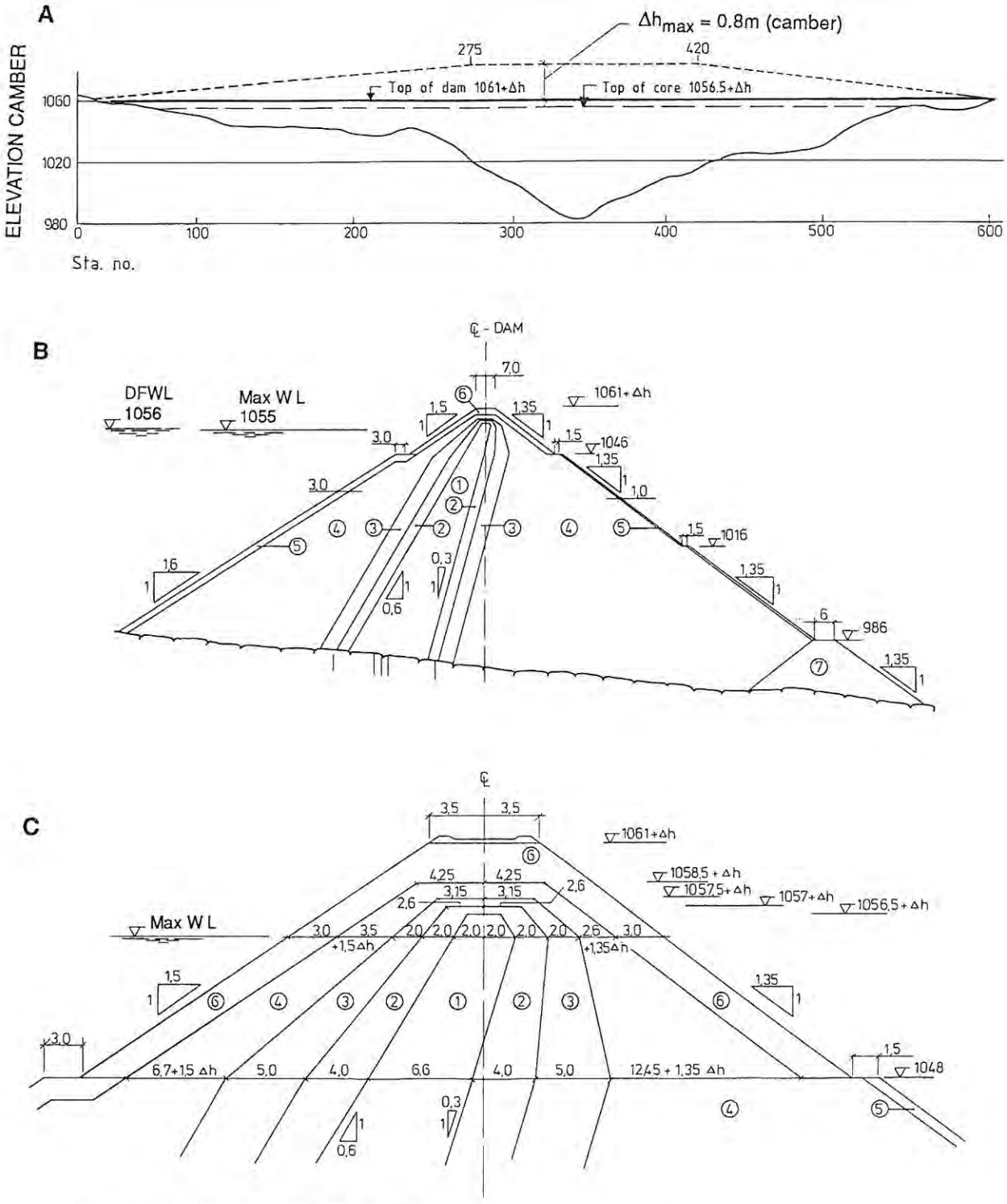


Fig. 6.2 Example of specification of cross-sectional dimensions and camber for a rockfill dam; Δh = camber

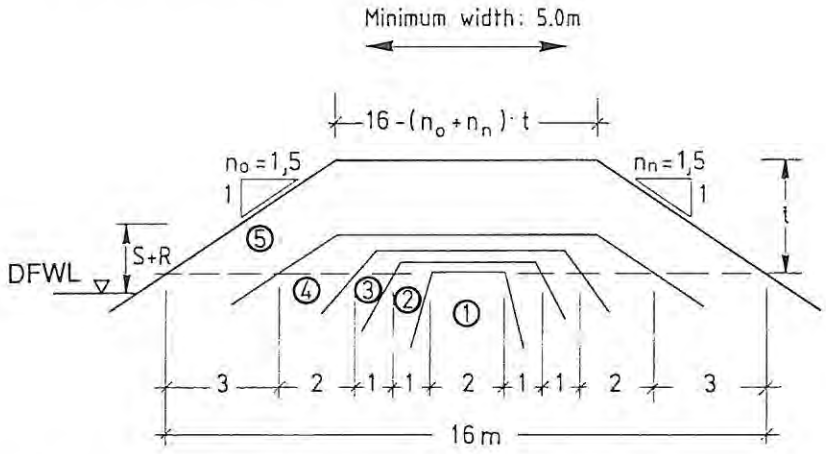
A) Longitudinal section B) Cross-section C) Crown details

without steepening the upper part of the slopes. It is preferable to assign the increase in width to the supporting shoulder fill, zone 4, and to keep the widths of all other zones independent of camber as illustrated in Figure 6.2.

For dams founded on compressible materials, the camber must cover total combined settlements of the fill and the underlying foundation material.

An increase in the width at the top of the dam, similar to an increase in freeboard, enhances the safety of the dam. The width at the top is, however, mainly dictated by practical considerations. It must be wide enough to allow proper placement and compaction of the fill and to accommodate planned traffic when the dam is completed. The width required may in practice depend somewhat on the length and the height of the dam, but usually is at least 5 metres. If a public road is involved, the relevant regulations must be observed and complied with. In any event, a road should be made at the top of the dam for inspection and maintenance.

In zoned rockfill dams with a moraine core and several zones of different gradings on both sides, the width is dictated by the working space requirements at the top of core level. Figure 6.3 shows an example of dam crown dimensions and construction lines. With the top of the dam 2 metres above the top of the core, the minimum width at the top of dam will be 10 metres. This is because, at the core top level, a minimum width of 16 metres is required to accommodate the nine separate zones.



SYMBOLS:

t	= min. 2.0m
DFWL	= Design Flood Water Level
n_o and n_n	= Upstream and downstream slopes
S + R	= Surge and wave run-up

DAM ZONES:

①	Core	④	Shoulder
②	Filter	⑤	Crown cap
③	Transition		

Fig. 6.3
Example of dam crown design resulting from the widths of the separate zones, slope inclinations and height of freeboard

If the height is increased by 2 m, the top of the dam would only be 4 m wide, which is insufficient in most cases. The width of the top of the dam

must therefore be set in accordance with the actual freeboard and working space requirements. General or special safety aspects and future traffic may require a wider crown.

6.6 Accidental Leakage and Required Drainage Capacity

Unexpected leaks through or under an embankment cannot be discounted. Filters which effectively prevent erosion are therefore of the greatest importance for the safety of embankment dams. The drainage capacity must be sufficient to let even large leaks pass through freely without creating high pore pressures or seepage forces which can be critical and threaten the stability of the dam.

Criteria for filter materials exist and are discussed in Section 2.3. They are generally based on the required grain-size distribution of the filter material related to the grain-size distribution of the base material to be protected against erosion. In view of their importance for dam safety, it is advisable to use a strict standard for the selection of filter materials as described in Chapter 2. The filter criteria should be met at each transition zone interface. Several filter zones may therefore be necessary to bridge the gradation gap between the earth core and the big stones in the slope protection and toe of the dam.

The drainage capacity denotes the maximum water flow intensity which the downstream fill can sustain without losing stability. The head of water required to cause the maximum flow intensity must not entail such rise of pore water pressures in the downstream supporting shoulder as to create deepseated sliding inside the downstream slope.

Similarly, the drainage capacity of the toe and slope protection indicates the greatest flow intensity which the toe and protection can sustain when

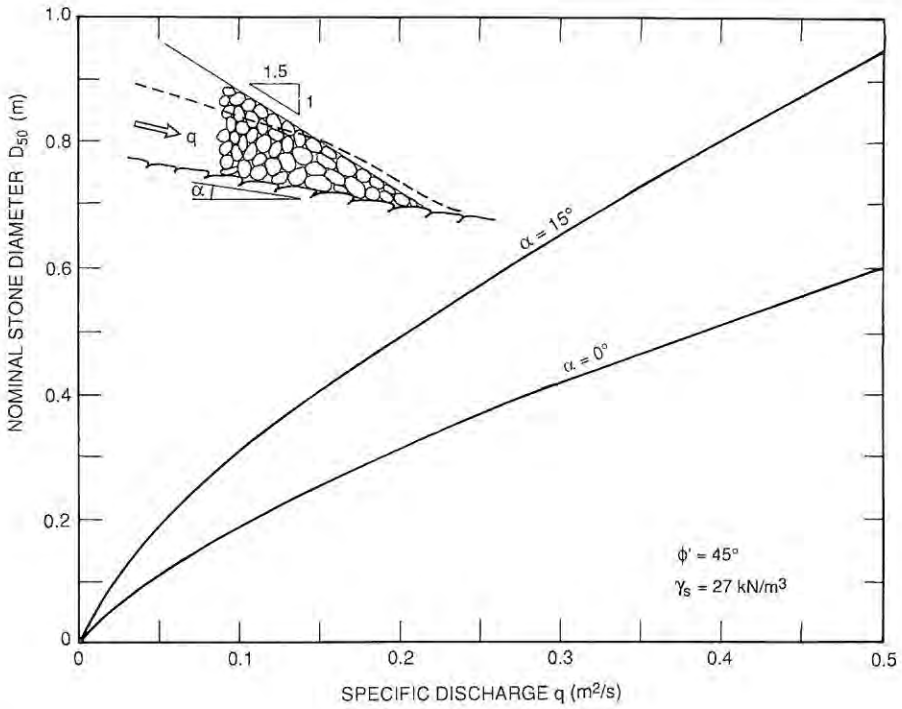


Fig. 6.4 Diagram for design of stones in the toe of the downstream slope (Modified by the authors from diagrams presented by Solvik, 1991)

subjected to flow without dislodgement of single stones from the toe. The critical flow intensity has been determined empirically from model tests. The diagram in Figure 6.4 gives the required size of stones in the toe for the appropriate outflow intensities. The size of stones depends on slope inclination, m, base inclination angle, α, effective angle of friction, ϕ', density of stones, ρ_r, and the shape of the stones.

The ϕ'-value is usually greater than 45°, but can be less on the interface for very smooth base rock surfaces. When exceptionally smooth or unfavourably inclined rock surfaces are encountered, the toe stones should be secured by using ditches excavated into the rock surface, concrete plinths, or similar measures to prevent sliding (Barton and Kjærnsli, 1981).

6.7 Upstream Slope Protection

Stones in the upstream slope protection must consist of sound rocks with sufficient weight to withstand the action of waves and ice sheets from the reservoir.

Design of rock armoured slope protection requires wave data for selected directions of wave approach. The wave data will be in the form of significant wave height, H_s, average wave period, T_z, and angle of approach to the slope (Chapter 4).

Design of armour stones has in Norway so far been based on the chart shown in Figure 6.5 giving the median weight of stones placed in an interlocking grid on the upstream slope protection layer. The design wave height, H_d, should be set equal to 1.3 H_s. The chart is based upon a formula developed by Hudson (1958) and experimental data from model tests of riprap on earth embankments (Thomsen et al., 1972).

Recent research has improved calculation of stone weights by taking into account the wave period and the wave breaking mechanism. A general

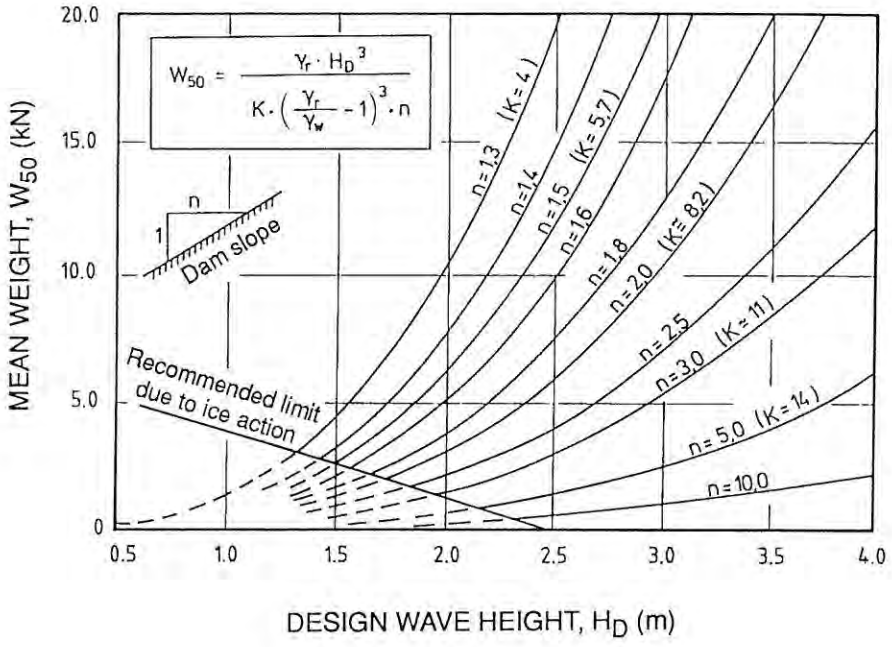


Fig. 6.5
Required weight of stones when individually placed in an interlocking pattern. Dumped stones must weigh twice as much.

stability formula for design developed by Pilarczyk (1990) can be expressed as follows:

$$\frac{H_s}{\Delta D_{n50}} = \frac{1}{F} \Psi K_M \cos \beta$$

where H_s : significant wave height (see Chapter 4);
$D_{n\,50}$: nominal diameter of median armour stone;
Δ : relative submerged density, $\Delta = \rho_s / \rho_w - 1$;
F : coefficient of safety against the threshold stability condition;
Ψ : system stability coefficient relative to randomly dumped armour rock;
K_M : stability number for randomly dumped armour rock;
ß : slope angle

The nominal diameter of the cover stones is defined as:

$$D_{n50} = \sqrt[3]{\frac{W_{50}}{\rho_s}}$$

where W_{50} : mean weight of individual stones, equal to the 50 per cent value of the grain-size distribution curve.

The stablity number, K_M , for randomly dumped armour rock is given in Figure 6.6 as a function of the surf similarity parameter (Meer, 1987). The diagram in Figure 6.6a is applicable for rockfill dams with a pervious fill below the armour stones, and the diagram in Figure 6.6b for earthfill dams with a shallow filter and an essentially impervious fill (silt, clay) below the armour stones. The surf similarity parameter, ξ_z, is defined as:

$$\xi_z = \tan \beta \; / \; \sqrt{\frac{2\pi H_s}{g T_z^2}} = 0.40 \tan \beta \; T_z \sqrt{\frac{H_s}{g}}$$

The stability of armoured rock can be improved substantially by placing stones in an interlocking grid rather than dumping randomly on the slope. In the stability formula this is accounted for by the system stability coefficient, Ψ. Typical values for individually placed stones are:

$\Psi = 1.0$: reference value for randomly dumped armour stones;

$\Psi = 1.3$: armour stones placed individually in an interlocking grid with the longest axis perpendicular to slope;

$\Psi = 1.5$: armour stones placed in a closely fitting dry masonry.

A Ψ-value of 1.3 can be applied for armour stones placed as illustrated on Photo 7.13.

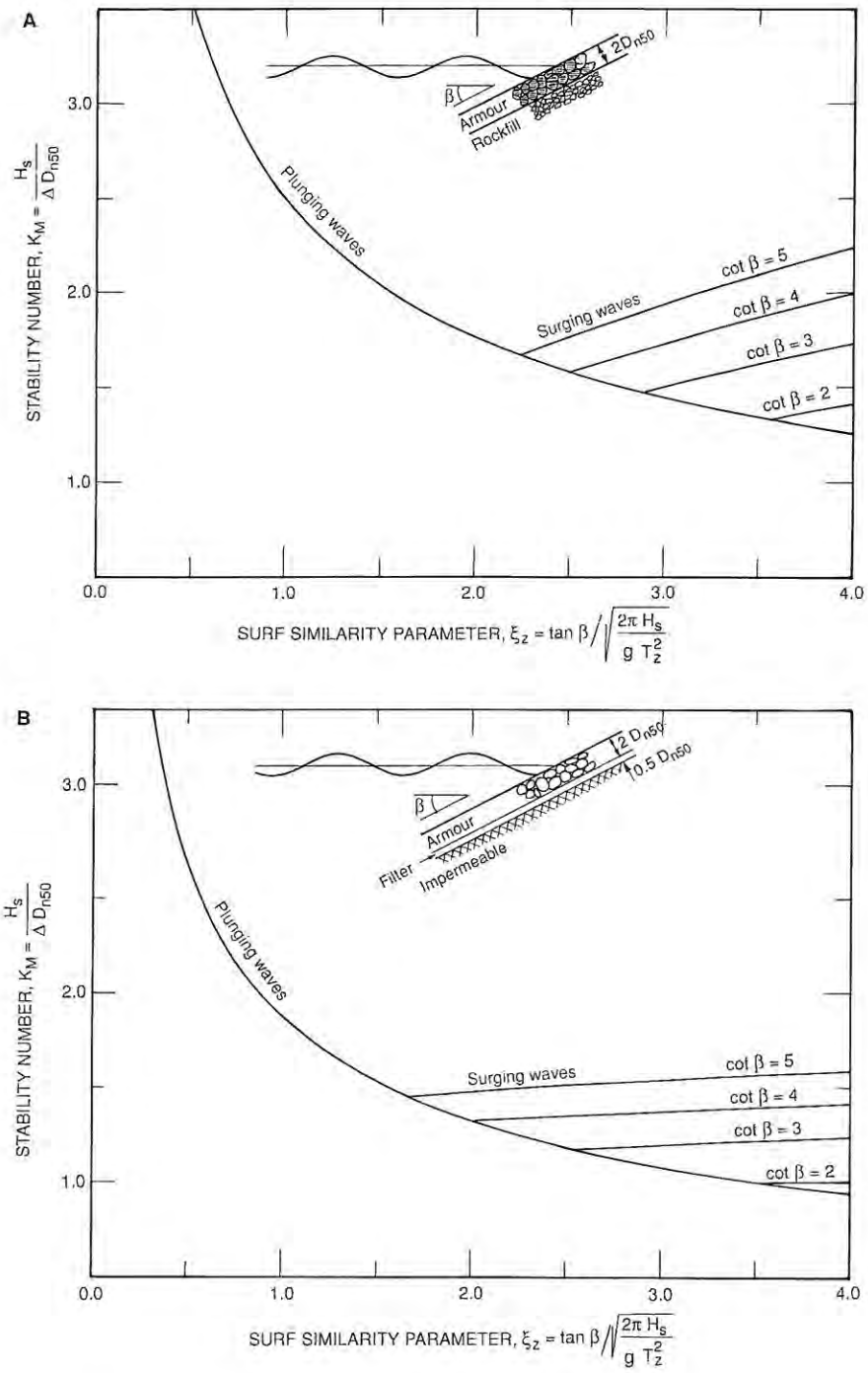


Fig. 6.6
Van der Meer's diagram for stability number, K_M, defining the stability threshold value for randomly dumped armour stones. (After Meer, 1987: using $S = 2$ and $N = 10\,000$)

A) Permeable base ($P = 0.5$)

B) Impermeable base ($P = 0.1$)

The procedure for using the general stability formula is:

1. Given the characteristics of the waves, H_s and T_z, and the slope angle, β, the surf similarity parameter, ξ_z, is calculated.
2. The stability mumber, K_M, is taken from the appropriate diagram.
3. The system stability coefficient, Ψ, is selected according to the placement technique to be applied.
4. The coefficient of safety against the treshold stability condition can for embankment dam slope protection be set to 1.1.
5. The nominal diameter, D_{n50}, and the corresponding median weight, W_{50}, is calculated from the stability formula.

Stones selected and individually placed in a slope protection cover shall be uniformly graded. The ratio, W_{85}/W_{15}, should be less than 10. For control purposes it may be simpler to stipulate the minimum weight instead of the median weight. The safest way may be to stipulate a minimum size equal to the median size determined. However, minimum weight of half the median can be tolerated.

It is impractical to weigh the individual stones. Stone size is characterized by the longest, shortest and intermediate cross-sectional measures of the stone. A given weight or volume of stones may be converted to a corresponding median diameter size by:

$$D = \sqrt[3]{\frac{W}{K \rho_s}}$$

where K: shape coefficient for quarried stones;
$K = 0.45$ for elongated stones typical for schistose rocks; and
$K = 0.75$ for cubical stones typical for granitic rocks.

The thickness of the upstream slope wave protection cover should be at least twice the largest stone diameter. The adjacent stone material must be coarse enough to meet the requirements of a base material for the given filter of large uniformly sized stones. Stones of a diameter less than about 1/10 of the diameter of the blocks in the slope protection cover may be washed out through the openings between the blocks.

On reservoirs with small wind fetch surfaces the design wave height would be negligible, and possible ice forces become critical for the dimensioning of stones in the slope protection cover. Figure 6.5 shows the stone sizes required to resist ice forces according to slope inclination as indicated.

6.8 Slope Stability Analyses

A slide in soil materials often leaves a more or less distinct sliding (slip) surface. The body overlying the slip surface moves relative to the underlying material that remains in place. Figure 6.7 shows a typical surface for a slide in a relatively homogeneous earth slope. The sliding surface in this illustration is simplified to a circle segment in a vertical section through the slide. The mechanics of the limiting equilibrium situation for this case is very simple, and the main uncertainties appear in the determination of shear strength along the potential sliding surface.

The safety factor against sliding is commonly defined as the number by which the shear strength is divided in the equilibrium analysis. The probability of a slide occurring depends on the mean safety factor and on the statistical scatter (coefficient of variation) of the shear strength for points along the sliding surface. With a mean safety factor of 1.0, a slide is highly probable, whereas a factor of 1.3 to 1.5 for embankments of fairly well known properties (i.e. coefficient of variation small), represents a very low probability of failure.

$$W \cdot a = \frac{s}{F} \cdot L \cdot r$$

W	Weight of soil
a	Lever arm
s	Shear strenght along failure surface
L	Length of failure surface
r	Radius of failure circle
F	Factor of safety

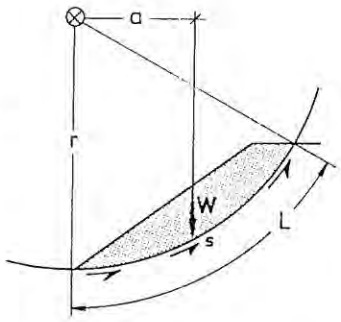


Fig. 6.7
Moment equilibrium assuming a circular sliding surface

Although the circular form is common for fairly homogeneous conditions, the sliding surface may take other forms according to circumstances. Janbu (1973) has given a comprehensive presentation of analyses for more general sliding surface configurations. Multilinear sliding planes should be studied, especially in a rockfill dam with distinct zones of different properties (Figure 6.8). Stability along multilinear sliding planes is analysed by means of force equilibrium equations, the so-called "sliding block" method (Figure 6.9).

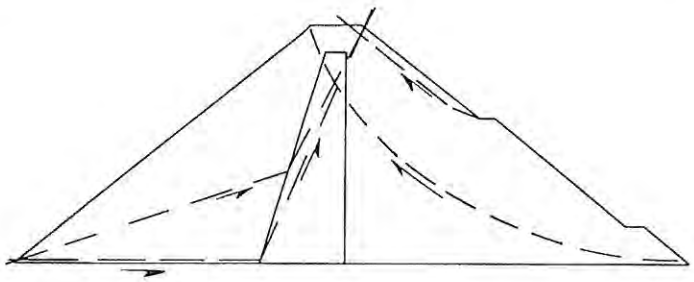


Fig. 6.8
Typical sliding surfaces used for calculation of the stability of a rockfill dam resting on bedrock

The shear strength along the different sliding surfaces are determined as described in Section 3.4. The strength parameters for the rockfill and filter, the core material and the rockfill-base interface must be determined as well as the effective normal stresses on the potential sliding surface. The latter requires estimates of total normal stresses and the pore water pressures at different points in the dam interior.

The stability of an embankment dam must therefore be analysed for several sliding planes in various dam cross-sections, and the numerical analyses are best carried out by means of special computer programs. Figure 6.9 shows potential sliding planes and corresponding "block" arrangements depending on the shear strength of the foundation as compared with that of the embankment. Stability is analysed for the highest of each type of cross-section along the dam axis.

The force vectors in the stability equilibrium equations result from material weight, water pressure as external load, pore water pressure on the sliding planes as internal loads, shear resistance along the sliding planes, and the effective normal forces on the sliding planes. The factor of safety is commonly defined as the number by which both c' and $\tan \phi'$, is divided in the limiting equilibrium analyses.

The external and internal forces for an embankment dam are not constant with time, and stability must be ascertained under various conditions. In practice computations are made for three design situations: end of construction, steady state seepage at full reservoir, and rapid drawdown of the reservoir level.

The end of construction and first impoundment is characterized by the transient pore pressures in the core not yet being adjusted to the impounded water regime. Pore pressures will rise with the increases in overburden load during the construction of the embankment. They will dissipate slowly in keeping with the consolidation properties and width of the core. The transient pore pressures are estimated on the basis of the anticipated embankment construction and reservoir filling rates using two-dimensional consolidation theory (Koppula and Morgenstern, 1982). The parameters needed for the computations, $\bar{B}$ and c_v, are determined from laboratory test results as described in Section 3.4.

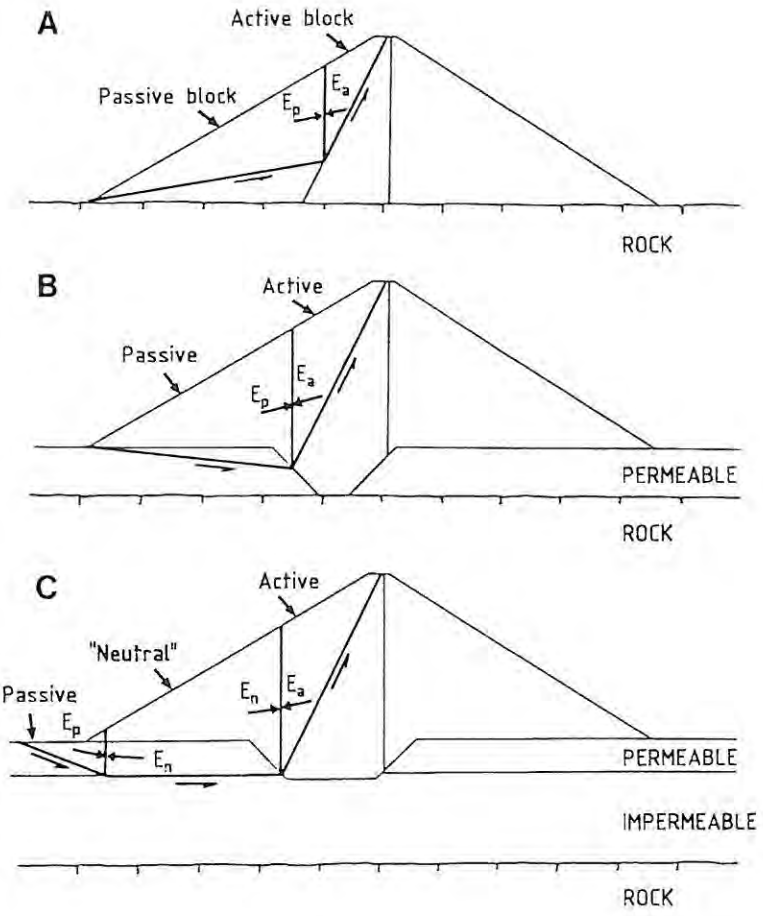


Fig. 6.9
Potential sliding planes and corresponding "sliding blocks" depending on the relative shear strength of foundation and embankment

A) Higher shear strength in the foundation

B) Higher shear strength in the embankment

C) Higher shear strength in supporting shoulder and permeable layer than in an impermeable layer of the foundation

External water pressure is introduced by using a value which corresponds to the water level at which stability is most critical. Internal pore pressures are estimated and may be monitored by piezometers installed during construction. If measurements indicate that too high pore pressure are developing, corrective measures must be taken. These measures may include slowing down the rate of construction, introducing relief drains, flattening outer slopes or reducing core width.

The condition of steady state seepage is said to exist when pore pressures at all locations are fully adjusted to the maximum water level regime. The pore water pressures are determined from flow net or finite element analyses with an external water head at the retension water level.

The condition of rapid drawdown is characterized by the transient pore pressure which results from a sudden drawdown of the reservoir from a maximum water level to a lower critical water level. This may give a critical stability situation. Pore pressures during the drawdown are set equal to the pore pressures for steady state seepage at maximum water level, less the changes in pore pressure due to reduced total overburden stresses. The external water pressure corresponds to the actual drawdown level.

Table 6.1 gives the safety requirements for rockfill dams in Norwegian practice. The table lists the various conditions for which stability analyses are required and the associated minimum factor of safety as defined above.

Under Norwegian design codes (limit state design), safety requirements are expressed through partial coefficients on load and material strength. For the types of stability analyses described above a load coefficient equal 1.0 is prescribed. The material coefficient therefore coincides numerically with the factors of safety listed in Table 6.1.

Table 6.1 Design conditions and required factor of safety against sliding

Design condition	Reservoir level	Sliding in slope	Minimum factor of safety
End of construction and first impoundment	Empty	Upstream	1.3
	Critical level (min. stability)	Downstream	1.5
Steady state seepage	High flood water level	Downstream	1.5
Rapid drawdown	Critical level (min. stability)	Upstream	1.3

6.9 Earthquake Effects

Norway is not in a very seismic region, and analyses have shown that the effects of probable earthquakes have not governed any aspects of the design of Norwegian rockfill dams (Valstad et al., 1991)

In regions subjected to stronger earthquakes, or for embankment dams and foundations more susceptible to earthquake shaking, stability and deformation analyses must include the effects of dynamic and cyclic (repetitive) loads. Due to the cyclic loading, excess pore water pressures may build up in poorly compacted fills or loose saturated foundations, leading to reduced shear strength, lower deformation moduli and the accumulation of perma-

nent (plastic) deformations (Andersen and Høeg, 1991). Strong earthquakes have in some cases caused complete liquefaction of foundations, slopes and embankments. However, compacted rockfill is inherently very resistant to earthquake shaking, and there is no record in the literature of damage to compacted rockfill dams caused by earthquakes.

Simplified stability analyses for dam embankments may be performed by introducing "statically equivalent" inertia forces (using seismic coefficients) into the equilibrium equations described in Section 6.8. The increased pore pressures induced by shaking, and any deterioration of the shear strength parameters, must be considered. It should be realized that the critical equilibrium stage may well occur some time after the earthquake shaking has ceased. Even if no instability and sliding take place, gradually accumulated displacements may have caused cracking to occur in the core. Analyses for such situations are presented in ICOLD (1986), Kuwano and Ishihara (1988) and Gazetas and Dakoulas (1992).

6.10 "Belt and Suspenders" Principle

The calculated safety factor or probability of failure is far from providing an unequivocal answer to the question of the overall safety of the dam. Theoretical calculations on their own can in no way offer a guarantee for the safety of the dam. This view was expressed so well by R. Peck in the 5th Laurits Bjerrum Memorial Lecture which he entitled "Where has all the judgment gone". An excerpt follows and concludes this chapter on design and analyses (Peck,1980):

..... a failure is seldom the consequence of a single shortcoming. Usually there is at least one other defect or deficiency, and the failure occurs where two or more coincide. This inference supports the principle of designing to provide defence in depth, the "belt and suspenders" principle long advocated by Arthur Casagrande. It postulates that if any defensive element in the dam or its foundation should fail to serve its function, there must be one or more additional defensive measures to take its place. Teton Dam is an outstanding example of violation of the principle; the sole line of defense was the core and grout curtain. Under such conditions, it is almost irrelevant to define the precise manner in which the failure started or whether there may have been such deficiencies in construction as a wet seam across the core. The design should have provided adequate defenses against any such defect.

The bedrock treatment appropriate to the geological conditions is a matter of design. It is not an aspect of design susceptible, however, to numerical analysis. Instead, it requires the exercise of judgment, a sense of proportion. When a jointed bedrock foundation is being treated and covered with the first layers of fill – a crucial time with respect to the future performance of the dam – engineers fully acquainted with the design requirements should be present, should have the authority to make decisions on the spot, and should not delegate their authority unless and until they are satisfied that their judgment concerning the particular project has been fully appreciated by their subordinates.

I doubt if guidelines, regulations, or even the best of specifications can take the place of personal interaction between designers and field forces at this stage. Frequently, as a consultant or as a member of a board of consultants, I have walked over a recently exposed foundation or abutment with the field forces, discussing place by place what treatment would be appropriate. Together we studied details of foundation conditions, observed treatment in progress, agreed upon or changed it. In this manner, the consultants evaluated the potential problems and their solutions, and the field forces gained the necessary insight as to what was required.

The type of foundation treatment is not a matter to be determined by a geologist, unless he is truly an engineering geologist. The geologist should investigate the geological characteristics of the foundation and its interface with the dam and make them known to the engineer who will form the judgment concerning treatment. The consequences of the flow of water near the interface, including its effect on the various materials in the dam, are within the realm of the engineer and the decisions regarding treatment are engineering decisions.

The three failures that I have discussed in some detail originated at the interface between dam and foundation. Others, that might equally have been chosen as examples, arose from overlooking or misjudging geological features in the foundation itself. A few had their origin in construction defects in the embankment. They had in common that all were outside the scope of numerical analyses. They would have been prevented if judgment arising from extensive experience had been given full scope in design and construction, and if the design had included multiple defenses. Accordingly, I think it quite possible that the incidence of failure of major dams could be reduced an order of magnitude by focusing attention on the details of design and construction that cannot at least presently be covered by analysis, that perhaps cannot be known until construction is underway, but that require the personal attention of experienced engineers. Research should be directed to those aspects of design and construction in greatest need of improvement; definition of foundation conditions; conditions conducive to internal erosion and the means for controlling it; filter criteria and their practical achievement in construction; prevention and treatment of cracking. The goal of the research should be improved understanding, preferably but not necessarily quantitative.

The literature already has much to say about cracking of earth dams. The emphasis, however, is on the mechanics of producing the initial cracks, an aspect that has recently become at least partly amenable to analysis. The analytical results serve a useful purpose: reduction of cracking can undoubtedly be achieved most successfully if the causes of cracking are understood and avoided. Nevertheless, to accord with the principle of defence in depth, every dam should be designed on the assumption that the core may crack and that the dam should be safe even if it does.

So we reckon with the conclusion that modern dams seldom if ever fail because of incorrect or inadequate numerical analyses. They fail because

inadequate judgment is brought to bear on the problems that, whether anticipated or not, arise in such places as the foundation or the interface between embankment and foundation. Sometimes they disclose themselves only in subtle ways in the use of the observational procedure. Irrespective of specifications, contractual arrangements, possibilities of claims for extras, or delays, these problems must be recognised and solved satisfactorily. If we regard them as second-rate judgment, failures of dams will continue at a probability of 10^{-4} per dam year. As long as the myth persists that only what can be calculated constitutes engineering, engineers will lack incentive or opportunity to apply the best judgment to the crucial problems that cannot be solved by calculation.

Where has the judgment gone? It has gone where the rewards of professional recognition and advancement are greatest – to the design office where the sheer beauty of analysis is often separated from reality. It has gone to the research institutions, into the fascinating effort to idealize the properties of real materials for the purposes of analysis and into the solution of intricate problems of stress distribution and deformation of the idealized materials. The incentive to make a professional reputation leads the best people in these directions.

From a probabilistic point of view, it is logical to assume a base level probability of failure of 10^{-4} per dam year. There is no reason, however, why engineers should be satisfied to consider such a failure rate as the norm. Dams should be designed and constructed not to fail, even if a probability of failure is incorporated into the benefit-cost analysis. Since we know wherein the greatest weaknesses lie, we should be able to devise the means for applying judgment to avoid these weaknesses. If we succeed, we should be able to justify a base level probability of failure no more than perhaps 10^{-5} per dam year. Such an improvement is now within the state of the art. Its achievement does not depend on the acquisition of new knowledge. It depends on our ability to bring the best engineering judgment to bear on problems that are essentially non-quantitative, having solutions that are essentially non-numerical. To develop this judgment and to bring it to bear requires a re-ordering of our present views of what constitutes the highest form of our practice of engineering. Without detracting from the necessity for reasonable and meaningful engineering calculations and from the rewards to those who can carry them out, at least equal professional prestige and responsibility should be accorded men of judgment, even when that judgment is not expressed in numerical form.

7.
CONSTRUCTION AND CONTROL PROCEDURES

Peck's statements in Section 6.10 emphasize the importance of construction and control procedures and the personal interaction between designers and field forces.

The construction work shall be executed in accordance with detailed plans, drawings and specifications prescribing the materials, construction and quality required for the work. The quality control shall be exercised in accordance with a specific program also listing the type and extent of sampling and testing. The control function shall be performed under the direction of a competent field inspector independent of both the contractor's and owner's field management.

The work specifications in dam construction largely prescribe means and methods rather than quality, which is only measurable to a limited extent. The control work will essentially consist of ensuring that the specifications are being followed and that the materials are consistently of the prescribed quality. Control of quality by regular testing will often be limited to the impervious elements and filters of the dam.

Execution and control of the separate operations in the construction of a rockfill dam are briefly described in the following. However, organization of the work, rigging and mobilization and geodetic surveying are not covered herein. The water level at the dam site is presumed to be taken care of by a cofferdam and an adequate diversion tunnel or a by-pass for water during the construction period, and this topic is not discussed.

7.1 Rock Foundation Treatment

This section focuses on treatment and preparation of rock foundations. Left-in-place overburden requires special treatment, and examples are presented in Figure 1.6 and in Section 6.2. The entire foundation area is stripped of topsoil and organic matter down to rock or firm ground such as gravel or moraine. Wherever reasonable, complete removal of the overburden is desirable for the construction of a rockfill dam.

As soon as the excavation is finished, the boundaries of the various embankment zones are marked on the rock surface. Cleaning of the foundation

for the supporting shoulders and transition zones is carried out mechanically, using backhoe excavators equipped with a special toothless bucket blade especially adapted for the purpose.

Excavation by blasting is carried out to smoothen the foundation under the core and filter zones or to remove rock of poor quality. At times blasting may also be necessary for roughening exceptionally smooth or steep rock surfaces under the embankment toe to ensure that the rockfill will not slip on the foundation (Section 6.6). Blasting must be adjusted to its purpose and local conditions. Smooth-wall blasting techniques are used for any blasting within the core and filter foundation, i.e. precise drilling, reduced burden, moderate charging and properly timed ignition sequences to avoid damage to adjacent rock. The field inspector decides where and how blasting is to be done and checks the results by observations during the subsequent wedging of the blasted surfaces. Loosened rock fragments in the core foundation are chipped off by chisel or wedge hammer and removed (Photo 7.1 at end of chapter). Final cleaning of the core foundation surface is done by jetting with water and compressed air.

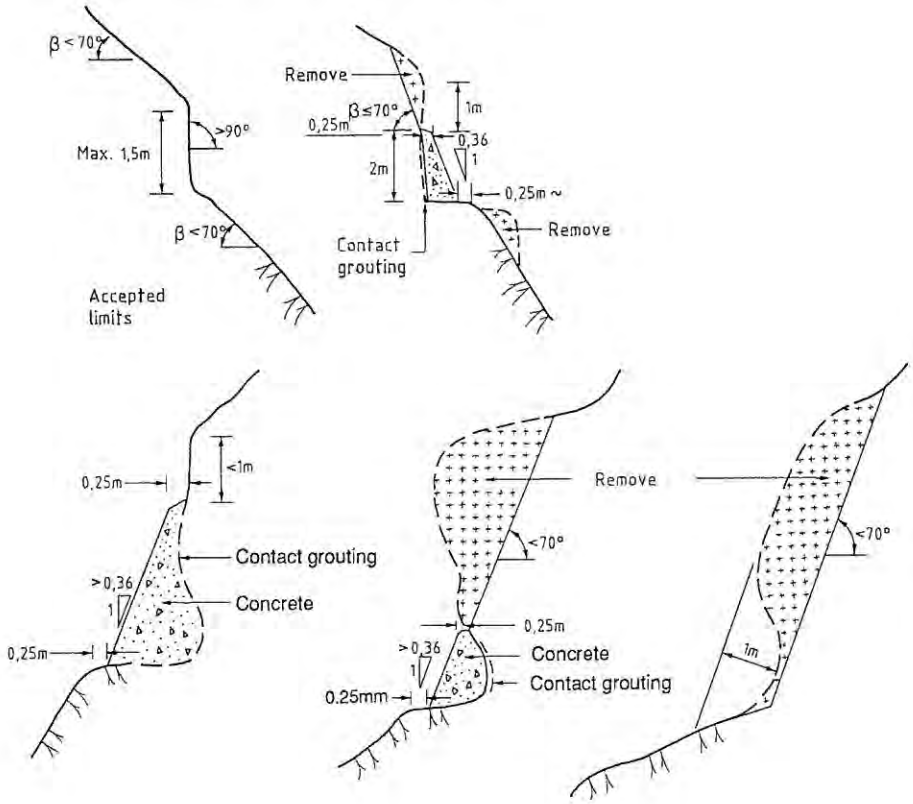


Fig. 7.1
Guidelines for smoothing of the core foundation cross-valley profile by blasting and concreting (After Peck, 1981)

Concrete work is an integral part of the preparation of the core foundation ("dental concrete"). Under earth cores concrete is used for smoothing profile steps and overhangs, filling clefts and sealing dykes, faults or poor rock. Sealing operations are often carried out with shotcrete (Photo 7.2).

Surface fissures are sealed with cement slurry applied by brushing just before laying the contact layer of moist core material. Figure 7.1 sets out general guidelines for acceptable smoothing of core foundations by blasting

and concreting. When employing impervious, thin diaphragms of various kinds, the above mentioned dental concrete will generally be substituted by a continuous reinforced concrete sill or plinth. Whatever method is employed, the concrete works include shuttering, reinforcement and rock bolting as needed.

The concrete work is generally subject to separate control. The field inspector's role is mainly to ensure that the work is executed in accordance with instructions and existing rules and regulations and that accepted practice and normal guidelines for the various applications of concrete are followed. The quality of the concrete work must not be jeopardized by flaws and shortcomings in the execution. Soiled forms or rebars, segregation or wrong consistency of the mix, lack of compaction and curing should not be tolerated. Concrete in the core foundation should be covered by core materials to a frostproof depth during winter construction shutdown.

7.2 Rock Grouting

The impervious element in the dam is extended in depth and width into the foundation to prevent excessive seepage through the ground underneath or adjacent to the dam. It is important to emphasize that grouting work should be done by specialists who have experience and knowledge of the various problems that may be encountered. The field inspector should have equal knowledge of the subject. His main task, however, is to control the results of grouting and ensure that the required watertightness is achieved (Houlsby, 1982).

Grouting work is carried out as surface grouting (shallow grouting) and curtain grouting (deep grouting). The purpose of surface grouting is primarily to secure water tightness to a relatively shallow depth underneath the core foundation area. Secondly it is to confine the subsequent high pressure grouting in the curtain. Surface grouting is therefore carried out prior to curtain grouting, since the curtain is situated in the center of the surface grouting.

The boreholes for grouting are located along rows parallel to the centreline of the impervious element of the dam. The number of rows and especially distances between holes must be appraised according to rock conditions. On both sides of a borehole where a water pressure test (Section 2.1) or the grout take indicate unsatisfactory watertightness, then additional holes are drilled halfway between neighbouring holes, "the split spacing method".

The need for rock watertightness will generally increase with the height of the dam. It will also decrease with depth below the foundation surface. For example, at shallow depths water pressure test values should be less than 1 to 2 Lugeon (L). At greater depths values of 2 to 4 L may be acceptable. Depending on the grout take, a corresponding standard of 5 to 10 kg cement per metre of borehole may be set. This may be used after some experience is gained on site and does not require a water pressure test. This will reduce costs and the chances of cement being washed out before it is set.

Cement grout has been successfully employed in improving most rock foundations for dams. Bentonite in the proportion of 2% of the cement weight is usually added to the grout. During the grouting process the water/cement ratio is gradually reduced from 4/1 to 1/2. If a thin mix is used, and the required grouting pressure is not reached, a thicker mix should be employed. When required pressure is achieved, it should be maintained for 15 to 30 minutes to force excess water out of the mix.

Upstage grouting, i.e. starting with the packer near the bottom of the hole when it is drilled to its final depth, and moving the packer upwards in stages, is the most common procedure. If, however, the holes prove to be unstable due to strongly jointed rock, then downstage grouting is adopted, i.e. starting by drilling to limited depth, grouting with the packer at the top of a hole, and then redrill the hole to greater depth after the grout has set.

Table 7.1 Grouting of some Norwegian embankment dam rock foundations – statistical summary

Geological environment	Statistical value	Curtain area / (max height x crest length) (m^2/m^2)	Borehole length per unit area of curtain (m/m^2)	Grout take per unit length of borehole (kg cement/m)	Grout take per unit area of curtain (kg cement/m^2)
Granitic and gneissose rocks (50 dams)	min max mean st dev	0.11 1.07 0.42 0.17	0.28 2.16 0.60 0.33	2.4 57.6 18.9 13.3	0.8 33.5 11.1 8.5
Amphi-bolitic and gabbroic rocks (8 dams)	min max mean st dev	0.23 0.68 0.38 0.14	0.44 0.95 0.59 0.17	4.2 67.8 24.8 19.3	2.0 41.6 15.0 12.5
Schists and phyllites (18 dams)	min max mean st dev	0.14 0.78 0.37 0.15	0.10 1.47 0.47 0.28	0.0 69.7 14.8 15.6	0.0 26.6 6.7 6.1
Meta-sand-stones (4 dams)	min max mean st dev	0.14 0.47 0.35 0.13	0.42 0.71 0.60 0.12	12.3 26.7 18.0 5.6	5.9 15.3 10.9 3.7
Limestone (1 dam)		0.28	1.34	18.3	24.5

Table 7.1 indicates the extent of grouting work executed for a number of Norwegian dams resting on rock foundations. On average, 10 metres of borehole and 150 kg of cement have been used for each linear metre of dam.

The grouting pressure should normally be as large as possible without disrupting the rock foundation. A grouting pressure measured at the top of the hole of 0.5 bar is regarded as a minimum. If the rock foundation does not tolerate this pressure with the packer placed just below the surface, the rock may either be bolted, or an anchored concrete stab may be placed over the area. Alternatively the packer may be placed at a depth allowing minimum pressure to be applied, followed by careful removal of the ungrouted overburden.

7.3 Borrow Pits and Haul Roads

Big machines offer obvious advantages in the actual operations which include excavation, loading, hauling, placing, spreading and compaction of materials. Some disadvantages due to size arise, however. These are: The need for wider haul roads and higher load capacities, more processing of the materials and wider filter and transition zones. Also needed is stricter control to ensure that the fill materials ending up in the dam are of even quality, correct grading and that they are in the right place.

Quarries and borrow pits represent a substantial part of the work associated with a rockfill dam. Opening borrows includes constructing access and haul roads, stripping and possibly draining the area. Collecting, draining and diverting surface and ground water is especially important in borrow pits for moraine core material.

Moraine is loosened by front digging excavators or bulldozers, possibly with a ripper. In layered deposits the excavation is done so as to facilitate mixing, either by a sloping trench cutting through the layers or by high wall front end excavations. In this case the excavator is used for both excavation and loading. Otherwise loading is usually done using wheel loaders (Photo 7.3). In gravel pits excavation and direct loading with shovel loaders is common. Gravel under water is generally excavated by backhoe.

Core- and filter material often have to be screened. Moraine is screened on a slotted grid with or without vibration. The slot openings are adjusted to the stone content. The lower the stone content the larger the opening should be. Segregation must be avoided by appropriate silo and discharge hopper arrangements. The screening plant must have sufficient capacity to meet peak production supply directly. Stockpiled core material easily absorbs water. It must therefore be placed on drained ground and either be shielded or so well compacted that water cannot permeate the mass.

Filter material is screened on vibratory meshed screens, which are easily adjusted to actual grain size requirements. The capacity should correspond to the average supply needed as material can be stockpiled without problem, apart from the need to avoid segregation.

Quarries should be located in suitable rock formations, preferably within the future reservoir and as close to the dam as possible without risking any damage or disturbance to other activities on the dam. Drilling patterns, drillhole diameters, type of explosives and size of blasting rounds are

adapted to produce the desired fragmentation. The quarry is usually opened by levelling the top with light weight drilling equipment. Then heavy equipment and large bore drilling rigs are put to work in the regular bench blasting operations.

In planning quarries or borrow pits, and in providing for necessary capacities in the various stages of operations, the need for expansion in volume must be taken into account. Fill materials are generally measured by volume. Volume, however, will depend on density of the mass in its present state. For moraine and gravel, density in the borrow pit and in the dam are more or less the same. One cubic metre in situ will occupy roughly the same volume in the dam. During processing and transport the same amount of moraine or gravel will expand to a volume of 1.15-1.30 m^3. The expansion factor of solid rock to rockfill is bigger. One cubic metre of solid rock will amount to 1.25-1.50 m^3 of rockfill in the dam depending on situation and compaction, and about 1.6 m^3 during processing and transport operations.

Loading and hauling is done by heavy wheel loaders and dump trucks of similar size (50-100 tons). The quarry must be located sufficiently far from the embankment construction area so that quarry blasting does not interrupt the operations. Haul roads and driveways are carefully planned and their construction geared to the progress of the dam construction and embankment levels. Cuttings for roads in the valley sides should not create large and abrupt breaks in the foundation profile. In or near the core zone cuttings should be of limited height and no steeper than 70°. Special consideration should be given to slope stability of cuttings in layered and fissured rock.

On very steep mountain sides tunnels have been used even for temporary driveways. Alternatively, major blasting and concrete works should be used to smoothen the profile as the elevation of the access is raised.

Driveways across the core during construction should be kept to a minimum. At the crossing, the earth core must be covered with a filter of sand or geofabric sheets, over which a load-distributing layer of rockfill or gravel is spread to about 1 m thickness. Filter and road materials must be thoroughly cleared away before spreading of core material can be resumed.

For dams with asphaltic concrete cores, driveway crossings should be made using special steel plate bridges spanning the core. Outside the core the compacted surfaces, as a rule, provide driveways to any point of the embankment. Apart from possible road ramps necessitated by differences in embankment levels, no special roadworks are required for transport on top of the fills.

In some cases it may be necessary to remove soiled or decomposed material or to roughen the roadway surface. The field inspector can decide at his discretion, what treatment is needed before regular filling can proceed.

7.4 Compaction Effect and Layer Thickness

Rockfill and gravel constitute the main ingredients in rockfill dams. They account for about 80% of total volume in dams with moraine cores and approaching 100% in dams with thin diaphragms. The quality of the fill depends to a large extent on the construction placement and compaction procedures.

Earlier rockfill used to be dumped in high lifts > 3 m and sluiced with water without any vibratory compaction. Sluicing (jetting) with water is an effective way of compacting gravel and rockfill, especially for rockfill with a high content of fines. The water jet is directed towards the dumping face or spreading area in front of the lift. Usually 1 m^3 of water is required for each cubic metre of fill at a minimum nozzle pressure of 5 bar. The water should hit the fill as a concentrated jet.

Introduction of heavy vibratory rollers (>15 tons) made effective compaction possible not only for gravel but also for coarse rockfill material. In addition sluicing is used when densely compacted material is required, for instance in dams with thin impervious elements. It is also used where there is an excessive content of fines in the rockfill material, or when the rock used in the rockfill loses strength on wetting and edges and corners of individual stones therefore break. The latter effect may be significant and should therefore be "taken out" of the rockfill prior to the reservoir filling.

Mechanical equipment and methods of compaction have led to placement methods which are in principle the same for all zones in the dam, except for the slope protection cover. The material is spread in horizontal layers and compacted with vibratory rollers. Layer thickness is of critical significance for achieved density, strength and deformation modulus. Limited thickness reduces the permissible maximum stone size, on the other hand, the benefits include better quality, reduced overfill of expensive filter and transition materials and improved access on the dam.

Vibratory rolling has an optimal effect at a certain limited depth beneath the surface, depending on type of material and weight of roller. Figure 7.2 shows density profiles for sand compacted with a 12.5 ton vibratory roller. The density profiles show that compaction had little or no effect near the surface. The optimal effect, i.e. highest density, is obtained at a depth of 0.4-0.5 m below which the effect rapidly diminishes.

In order to attain an optimal degree of compaction throughout, the layer thickness must not be much more than the optimal depth. With greater layer thicknesses compaction will have little or no effect on a stratum around the layer boundary. Thus, layer thickness will be determined by the degree of compaction required for the fill (compare displacements measured at Svartevatn and Storvatn dams, Chapter 5).

The compaction effect depends on the frequency and amplitude of the vibrations which have to be adapted to the type of material. Vibratory rollers for use on coarse rockfill have lower frequencies and greater amplitudes

than have rollers for use on finely graded, cohesive materials. The travelling speed of the roller is an important factor, as compaction increases with decreasing speed. A few passes at low speed have the same effect as many passes at high speed. In practice 6-8 passes at speeds around 2 kilometres per hour produce about the optimal effect and efficiency for vibratory compaction of granular materials. Ideally, prior to embankment construction, test fills should be used to determine optimum compaction procedure wrt. layer thickness, type of equipment, number of passes, etc. However, this is a fairly expensive undertaking.

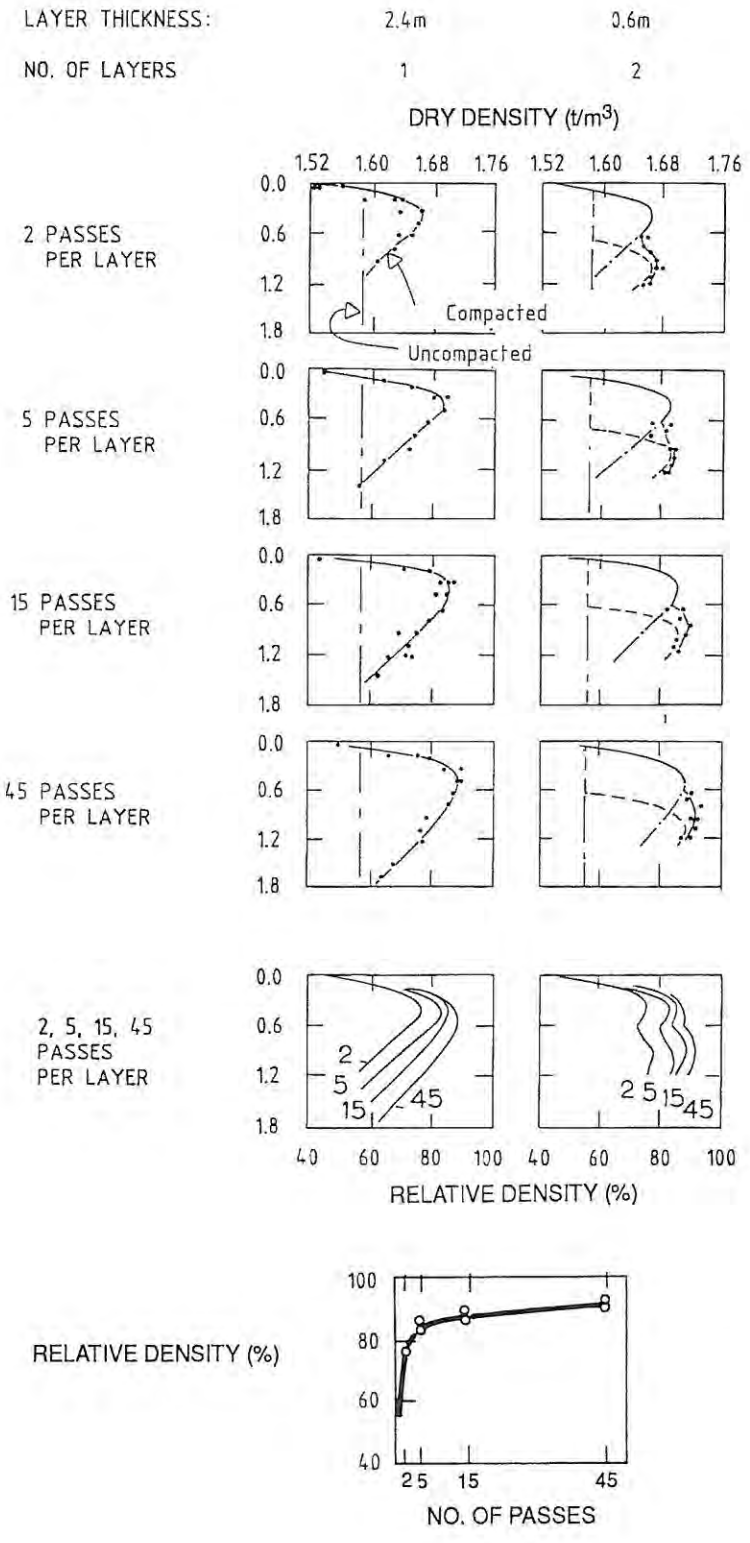


Fig. 7.2
Density profiles for sand compacted with a 12.5 t vibratory roller (After D'Appolonia et al. 1969)

In coarse rockfill, control of compaction is difficult and time consuming. Measurements of density and settlement during compaction or plate bearing tests (Figure 7.3) require elaborate testing apparatus and careful planning and execution to produce reliable results. Therefore, instead of specifying the degree of compaction (density) required, layer thickness, number of passes and roller weight are most commonly prescribed.

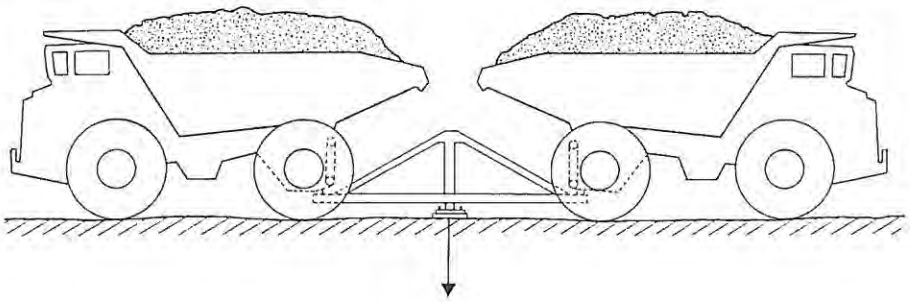


Fig. 7.3
Arrangement for plate bearing tests. Two loaded dumpers serve as counter weight on the loading yoke which transfers the load to the plate through a hydraulic jack.

Direct control of the relative degree of compaction has been made possible with the introduction of so-called compactometers (Figure 7.4). The compactometer receives a signal from an accelerometer mounted directly on the frame of the vibratory roller. The numerical display or printer pattern shows whether the sub-base has attained a uniform and high deformation modulus along the path of the roller. The compactometer is therefore useful in providing information on the relative quality of layers of varying thickness, roller speed and number of passes.

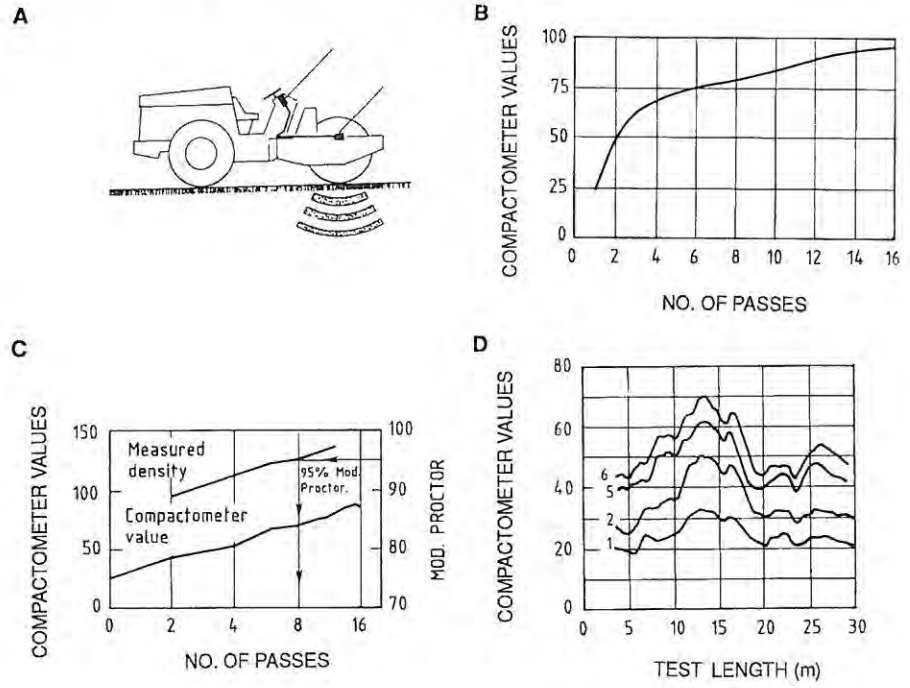


Fig. 7.4
Vibratory roller with compactometer

A) Compactometer assembly on roller

B) Compactometer readings over number of passes

C) Calibration testing: Compactometer readings and measured density over number of passes

D) Compactometer readings registered during each pass of a 10 tons tandem roller on a 30 m long stretch of varying sub-base quality (After Forssblad, 1981)

Another method for controlling compaction of granuar materials uses surface wave velocity measurements, and this technique should be developed further for practical applications (Madshus and Westerdahl, 1990).

It is important to keep the different zones at more or less the same level during construction, and layered zones should be raised in sequence with

each other. It may have negative consequences both technically and economically if widely different elevations of fill are allowed to develop within or between the various zones. In some cases deviations cannot be allowed. For instance, the raising of transition zone, filter and core zone layers must follow in that order. Otherwise, the prescribed treatment and compaction of the interface contact between zones cannot be carried out.

Similarly, the slope protection cover has to follow closely on the raising of the embankment so that armour stones can be placed individually with proper contact and orientation in accordance with work specifications.

In layered zones maximum stone size should be limited to $\frac{1}{2}$ to $\frac{2}{3}$ of the layer thickness. The fill is usually dumped on top of the advancing layer and spread by bulldozer on to the previously compacted layer. To avoid separation, the bulldozer must push the full heap in front of the blade in one sweep past the edge without lifting the blade.

In narrow zones, such as filter and transition zones, backhoes rather than bulldozers are used to place and spread the material. In this case segregation is used in a controlled manner to secure good interface transitions. Coarser material is systematically shifted from the inner to the outer side of the zone. To enhance interaction between zones and to control the filtering efficiency, transition interfaces are watered as necessary.

7.5 Earth Core Material

Impervious core material, such as moraine, should have a fines content of at least 15% of dry weight, and in general the water content should not deviate more than 2 percent points from the optimum. Somewhat larger deviations may be allowed provided the required degree of compaction still is attained (Section 3.1).

Placement and compaction of impervious core materials require fairly dry weather. In practice, the water content in impervious materials cannot be reduced, and they will have to be used essentially as they exist in the borrow pit. Artificial drying is very costly. With a water content approaching saturation, the effect of compaction is significantly reduced. With a low water content a high amount of compaction work is needed to achieve the desired properties. The situation may be rectified by sprinkling with water during processing. To a limited extent, it is also possible to do this during the spreading of the materials on the dam. It is essential that the masses become moistened to a controlled degree.

The correlation between water content, compaction effort and resulting density for a morainic material is shown in Figure 3.4. Vibratory roller compaction represents a higher compactive effort than the Proctor standard compaction, and the optimum water content for such compaction is lower than the Proctor standard optimum. With increasing water content the effect of vibratory compaction decreases more rapidly than with Proctor standard compaction.

Beyond a certain water content vibratory compaction becomes impracticable. However, it may still be possible to achieve some degree of compaction by means of a track-type tractor of suitable weight and track pressure. Tractor compaction will, however, have no effect beyond a shallow top layer and is therefore suitable only for comparatively wet material spread in thin layers of less than 0.25 m.

A main purpose of compaction is to minimise core settlements and reduce the risk of cracking. Whereas permeability primarily depends on the size of the pores fixed by the fines, long-term settlement will essentially depend on the total pore volume in the mass. The pore volume is determined from the dry density obtained during compaction. Compaction requirements for impervious core materials are usually specified in relation to Proctor standard compaction. The Proctor standard alone, however, does not give a fully representative picture of the higher compaction effect obtained with vibratory compaction.

At low water contents vibratory compaction gives a lower pore volume than at Proctor standard optimum. The pores are filled with air and water and only the air volume is reduced by compaction. Thus, a mass with less water and more air may be compacted to a higher density. Independent of water content, an upper limit may be set for the air volume in the compacted mass. The air volume should not exceed the air volume at approximately Proctor standard optimum water content. Consequently, total pore volume will be lower and the dry density higher with reduced water content. The pore volume cannot, however, be reduced beyond the minimum set by grain-size distribution and the shape of particles in the mass. The possible air volume limits are thus dependent on the type of material and are set accordingly.

The water and fines content together with particle shape and grain-size distribution determine the applicable layer thickness and method of compaction of the core. As a rule, moraine is placed in 0.5 m layers and compacted in 6-8 passes with a 10-15 ton vibratory roller. If the fines content is relatively high, say more than 50%, a smaller layer thickness may be necessary (Photo 7.4). If the water content is also on the high side, it may be necessary to reduce layer thickness to, say, 0.25 m and use a tractor instead of a vibratory roller for compaction (Photo 7.5). Core material which has become too wet during construction, must be removed and dumped. Artificial drying would be too expensive and natural consolidation takes too long.

Core material next to the rock or concrete foundation should be relatively wet and placed to a maximum of 10 cm thickness (Photo 7.6). Stones of 3 cm diameter or more are taken out either by screening beforehand, or by raking on the spot. The contact layers are compacted with suitable vibro plates or tampers which can be operated in confined places and corners (Photo 7.7).

Besides continuous visual inspection of the appearance of the material and the filling procedures, the core material is checked by systematic testing.

The fines content and water content in the core material are tested daily in samples of uncompacted material. The degree of compaction is tested once a week by digging test pits 1.5-2.0 m deep. Undisturbed samples are taken at various elevations in the wall of the pit. The density of each sample is measured in the laboratory. These samples are also tested for water content, dry density, specific density of solids, permeability and grain-size distribution. All laboratory tests should be performed in accordance with standardized procedures, and the test results presented on specific, standardized forms.

The full range of grain-size distribution of the core material is determined from large samples (2-10 ton) several times each construction season, depending on how much the grain-size distribution varies in the borrows from which the material is taken.

7.6 Filter, Transition and Rockfill

Requirements to filter and transition materials are presented in Section 2.3. Filter zones are usually spread to the same layer thickness as the adjacent core. The spreading is done by bulldozer or backhoe. A backhoe offers extra security against unwanted segregation as the material may be distributed selectively and agglomerations of stones may be shifted or removed as need be (Photo 7.8). Compaction of filter zones is done by vibratory roller, the number of passes being adapted according to the weight of the roller, layer thickness and type of material. Most often medium weight (6-8 ton) or heavy weight (> 15 ton) rollers are used in 2-4 passes.

Watering may be necessary or desirable, especially along the interface towards the transition zone and steep rock surfaces. Watering is done with free flow from an open hose. Watering cannot be done in freezing weather. Frozen materials must not be filled in or covered up in the filter zone (Photo 7.9).

The filter zones are basically controlled by visual inspection of the material and filling procedures. The grain-size distribution is checked frequently through screening tests. The degree of compaction obtained in the filters is not usually controlled but is occasionally checked in the beginning to obtain experience with the materials used.

Transition zones are often built in layers twice as thick as in the adjacent filters. The transition zone constitutes a secondary filter which is treated much in the same way as the primary filter. Spreading is done by bulldozer and/or backhoe. To avoid agglomerations of stones along the inner edge of the zone, the transition material is deliberately spread with an overlap on to the filter. The surplus material is subsequently excavated back, and shifted to the outer side of the zone, by backhoe (Photos 7.10 and 7.11).

Transition zones are compacted in 4 to 6 passes with a 10 to 15 ton vibratory roller. If necessary, the outer side towards the adjacent rockfill is sluiced with water.

The control of the transition zone consists mainly of close inspection during execution. The grain size distribution is inspected visually and checked with a moderate number of screening tests. The test samples must be fairly large, for instance 5 to 6 tons. Compaction is checked with a few density tests each construction season.

The rockfill of the supporting shoulders of the dam is spread by heavy bulldozers. The layer thickness is adapted to the adjacent zone and may be up to 2 m for compaction with heavy vibratory rollers > 15 ton (Photo 7.12). 6 to 8 passes would be required. Sluicing may be needed if the rockfill has a relatively high fines content, or when the rock used in the rockfill loses strength on wetting. The effects of layer thickness, vibratory compaction and water sluicing on in-situ rockfill deformation modulus are demonstrated in Chapter 5.

The slope protection and crown cap consist of selected, single size stones which are individually placed by backhoe with a bucket especially adapted for the handling of single stones (Photos 7.13 and 7.14). The armour cover stones are placed in an interlocking pattern with suitably close contacts between adjacent stones. In the slope protection layer the stones are properly oriented with their lengths approximately perpendicular to the slope. The control of slope and crown protection consists of ascertaining that frost and weather resistant stones of sufficient size are placed in the prescribed manner (Section 6.7).

7.7 Asphaltic Concrete Core Walls

Impervious core walls of asphaltic concrete are in many cases the best alternative to earth cores. The asphaltic conrete is compacted at a temperature of 160-180°C and is given immediate lateral support. Placement of the core wall and adjacent zones proceeds simultaneously, with equal layer thickness, usually limited to 0.20 m. The width of the asphaltic core is least 0.50 m. The adjacent zones are often narrow, and the small lateral dimensions require a high degree of precision in the execution of the work. Table 7.2 summarizes some key figures for Norwegian dams constructed with asphaltic concrete core and for two which are under design and construction (Höeg, 1992).

For asphaltic concrete cores special laying machines are used. The machines handle the core and adjacent gravel material simultaneously, discharging the asphaltic concrete immediately ahead of the gravel (Photos 7.15 to 7.17). The layers are levelled to proper thicknesses by the travelling machine which simultaneously gives an initial compaction to the materials (Photo 7.18).

Final compaction is achieved by vibratory rollers following the laying machine. The rollers should work in a co-ordinated manner, side by side, so as to avoid displacement of the hot asphaltic concrete layer (Photo 7.19). For the production of the asphaltic concrete, a controlled mixing plant is needed, complete with silos, heating drums, screening and weighing equipment. Special insulated hoppers are required for hauling the hot mix, which is

produced in strict compliance with the specifications for grain-size distribution of the concrete aggregates, filler and bitumen content in the mix, and the temperature constraints at the various stages of the process (ICOLD, 1992). The flexibility, cracking resistance and "self healing" ability may be improved by increasing the bitumen content and by reducing bitumen viscosity.

Table 7.2 Norwegian rockfill dams with asphaltic concrete core

Name	Height above lowest core foundation (m)	Core thickness (m)	Core inclination (v:h)	Vertical projection area of core (m^2)	Volume of asphaltic concrete (m^3)	Construction period	Main contractor/ Asphalt contractor
Vestredals-vatn	32	0.5	1:0	6000	3100	1978-80	Furuholmen/ Strabag
Katlavatn	35	0.5	1:0	4600	2300	1979-81	Furuholmen/ Strabag
Langavatn	22	0.5	1:0	3800	1900	1979-81	Furuholmen/ Strabag
Storvatn	90	0.5 0.8	1:0.2	76000	49000	1981-87	Statkraft/ Veidekke
Riskalvatn	45	0.5	1:0	14600	8000	1983-86	Veidekke/ Korsbrekke and Lorck
Berdalsvatn	62	0.5	1:0	13000	6800	1986-88	Veidekke/ Korsbrekke and Lorck
Styggevatn	52	0.5	1:0	30400	15300	1986-90	Statkraft/ Korsbrekke and Lorck
Storglomvatn (under construction)	125	0.5 1.0	1:0	~44000	~20000	1992-	Statkraft/ not decided
Holmvatn (under design)	56	0.5	1:0	~12000	~6200	1995-	Statkraft/ not decided

Adequate compaction and proper bonding between layers is of critical importance for the watertightness of the asphaltic concrete wall. The surface of the core must be clean and dry when the next layer is placed and must not be soiled in any way during the process. Drill core samples are taken to control the compaction. The density obtained is determined, and the air volume is calculated on the basis of the specific density of solids in the sample. The drill hole is subsequently backfilled with compacted asphaltic concrete or occasionally with hot bitumen mastic.

In Norway asphaltic concrete has mainly been used in central cores. In the Venemo Dam, however, asphaltic concrete was laid in an upstream deck. The deck was laid in 2-3 layers in sectional strips, starting at the bottom and proceeding up the slope. The placing and compaction equipment was operated by winching from the top (Photos 7.21 and 7.22). Production and inspection procedures for the asphaltic concrete in frontal decks are essentially the same as for central cores.

7.8 Stone-Bitumen Cores

Dams are also built with stone-bitumen cores which consist of uniformly graded crushed stones or pebbles impregnated to void saturation with bitumen. The core is built in consecutive horizontal layers. Metal sheet shuttering, which may be left in the dam, is used along the sides of the core wall. The layer form is filled with clean and dry stone material which must be accurately levelled in a horizontal manner over the entire length of the layer before penetration with hot bitumen can begin. Hot bitumen is pumped from a heated tank. The bitumen must flow forward as an advancing slope, and the hose nozzle is moved forward, in small steps, as the voids are filled to saturation. Care must be taken that no water or air is engulfed and trapped by an overleaping bitumen flow. Similarly, care must be taken to avoid any leaks or drainage of bitumen from the shuttering. That would entail the danger of hidden voids being left undetected beneath more rapidly cooling bitumen (Photos 7.23 and 7.24).

With a stone-bitumen core the adjacent filter zones become of special importance as they must be impervious to bitumen. The filter must be of such consistent fineness and compactness that bitumen cannot be squeezed out at any point by the water pressure.

The control of gradation and compaction of both the filter and transition zones, therefore, becomes especially important. The field inspection of the stone-bitumen core consists largely of ascertaining that the stone material is clean, that bitumen of correct grade and temperature is used, and that the penetration is carried out as prescribed. Testing probes are carried out where any suspicion of voids may exist.



Photo 7.1
Hydraulic hammer mounted on an excavator for chiselling away loose fractured rock and for smoothing surfaces (Innerdalen Dam, 1981 – Kraftverkene i Orkla)



Photo 7.2
Shotcreting of core foundation rock to smooth out small ledges and seal off cracks and fissures (Innerdalen Dam, 1981 – Kraftverkene i Orkla)



Photo 7.3
Borrow pit for moraine core material. The moraine is loosened by a bulldozer on a steep slope to ensure proper mixing in a layered deposit. The moraine is loaded by wheel loader. (Svartevann Dam, 1975 – Sira-Kvina Kraftselskap)



Photo 7.4
Compaction of moraine. Moraine is spread in 0.5 m layers and is here being compacted in 8 passes with a 10 ton vibratory roller. Two roller units are at work. One is self-propelled, the other towed by a bull-dozer. (Oddatjørn Dam, 1980 – Ulla-Førreverkene, Statkraft)



Photo 7.5
Track-type tractor compaction. Moraine is spread in 0.25 m layers which are compacted in 6-8 passes with a 15 ton bulldozer. The method is used on material with high fines content and relatively high water content. (Sysen Dam, 1978 – Eidfjordverkene, Statkraft)



Photo 7.6
Filling of impervious materials against rock or concrete; layer thicknesses 10 cm. Stones > 3 cm are removed either by screening beforehand or by raking on the spot. On level stretches compaction can be done with rollers. On steeper gradients or uneven surfaces vibro plates or tampers are used. (Svartevann Dam, 1973 – Sira-Kvina Kraftselskap)



Photo 7.7
Placement of impervious material against rock, here partially treated with shotcrete. (Oddatjørn Dam, 1980 – Ulla-Førreverkene, Statkraft)



Photo 7.8
Trimming of filter and transition material with a backhoe gives better quality than bulldozing. Stone agglomerations can be avoided and zone boundaries kept to more accurately. (Innerdalen Dam, 1981 – Kraftverkene i Orkla)



Photo 7.9
Sluicing of transition material during spreading. Segregation, as in the picture, results in the collection of larger stones along the ridge on the filter. The segregated material is excavated back and shifted to the outer side of the transition zone. (Førsvatn Dam, 1978 – Tokkeverkene, Statkraft)



Photo 7.10
Particle segregation is exploited here to obtain a good transition between the filter material and the adjacent tunnel spoil on the outside. The tunnel spoil is spread deliberately with its inner edge overlapping the filter by about half a metre. The overlapping surplus is excaveted back and shifted to the outer side of the tunnel spoil zone. Thus, only the finer fraction of the tunnel spoil remains adjoining the filter. (Svartevann Dam, 1975 – Sira-Kvina Kraftselskap)



Photo 7.11
Shifting of materials. Stones are excavated back from the edge along the filter boundary. (Førsvatn Dam, 1978 – Tokkeverkene, Statkraft)



Photo 7.12
Compaction of rockfill with a 15 ton vibratory roller. The vibration results in a rearrangement of the rockfill so that voids are filled with fine material. The rockfill on the surface is clearly crushed by the roller. Subsequent rearrangement occurs during compaction of the next layer. On excavation into compacted rockfill no special horizon has been found at the layer interfaces. (Svartevann Dam, 1975 – Sira-Kvina Kraftselskap)



Photo 7.13
Placement of block stones in the slope protection zone. (Nerskogen Dam, 1981 – Kraftverkene i Orkla)



Photo 7.14
Termination of the crown cap. Selected blocks (here covered by snow) are set at each edge of the top surface and aligned with the curvature of the dam. The roadway has been levelled with gravel. (Innerdalen Dam, 1981 – Kraftverkene i Orkla)



Photo 7.15
Placement machine for asphaltic concrete core and gravel filter zones. A backhoe excavator is used for traction and a crane to transfer the asphalt mix. Another excavator is used to feed the gravel bins at the rear. The machine has an automatic direction and level guidance system. (Storvatn Dam, 1985 – Ulla-Førreverkene, Statkraft)



Photo 7.16
The asphaltic concrete core is placed manually at the important junction with the foundation plinth. (Riskalvatn Dam, 1985 – Hydro Aluminium)



Photo 7.17
Laying machine for asphaltic concrete core. At the front is equipment for heating the previously placed layer. Then a shield separating the core from the gravel for the filter zones. The asphalt is laid through the funnel in the centre of the machine. (Vestredalsvatn Dam, 1981 – Oslo Energi)



Photo 7.18
Initial compaction of asphaltic concrete core and gravel filter zones at the rear of the placement machine. (Storvatn Dam, 1985 – Ulla-Førreverkene, Statkraft. Photo by A/S Sigurd Hesselberg)



Photo 7.19
Final compaction of asphaltic concrete core and gravel filter zones by tandem vibratory rollers. (Storvatn Dam, 1985 – Ulla-Førreverkene, Statkraft. Photo by A/S Sigurd Hesselberg)



Photo 7.20
Placement machine for asphalt concrete core and gravel filter zones. The filters are placed immediately behind the core to provide the lateral support necessary for compaction of the hot asphaltic concrete. (Riskalvatn Dam, 1986 – Hydro Aluminium)



Photo 7.21
Placement of frontal deck of asphaltic concrete. The placement is done in strips with an asphalt paver specially equipped for working on a slope. Joints between strips are reheated and recompacted. (Venemo Dam, 1963 – Tokkeverkene, Statkraft)



Photo 7.22
Coring samples from the asphaltic concrete deck for quality control. Sample diameter is 10 cm. (Venemo Dam, 1963 – Tokkeverkene, Statkraft)



Photo 7.23
Core wall of stone-bitumen. Stones placed in metal sheet shuttering before pouring the hot bitumen. (Brynsvatn Dam, 1981 – Nordkraft)



Photo 7.24
Core wall of stone-bitumen after pouring of hot bitumen. (Brynsvatn Dam, 1981 – Nordkraft)

8.
INSTRUMENTATION AND PERFORMANCE MONITORING

Evaluated experience and judgment combined with design analyses and construction control should together result in a dam structure of the desired quality and safety level. Even if these requirements are met, all large dams should be monitored. This is of special importance during the first filling of the reservoir.

The rate of reservoir increase may critically affect the development of potential cracks in the core. A slow and gradual filling of the reservoir is less taxing to the dam than a rapid filling, as the dam structure has time to adjust to the changes in loading. Controlling the rate of filling will often depend on the ability to release water from the reservoir, and the release capacity is therefore essential for the real safety of the dam.

Besides being of importance for the particular dam in question, data from instrumentation programmes also yield improved knowledge about dam behaviour in general. The programme must thus be assessed in view of the requirements for monitoring and control, the extent to which the data will yield improved knowledge to the benefit of future structures, and of course, what it will cost (DiBiagio and Kjærnsli, 1985).

8.1 Seepage Measurements

The effectiveness of the impervious element can be measured directly by the quantity of water discharge passing through the dam. Observations over several seasons yield not only the relationship between seepage and reservoir head, but also long term trends in seepage over time. Increasing seepage over time calls for special evaluation of the dam (Figure 8.1). Seepage water which is murky with dispersed soil particles calls for immediate attention as it might indicate a process of internal erosion.

In order to measure seepage, water emerging on the downstream side of the core should be collected at one or several locations. To avoid interference from precipitation or snowmelt the seepage should be collected inside the embankment immediately downstream of the core. As example, Figure 8.2 shows the layout for seepage measurement at Svartevann Dam.

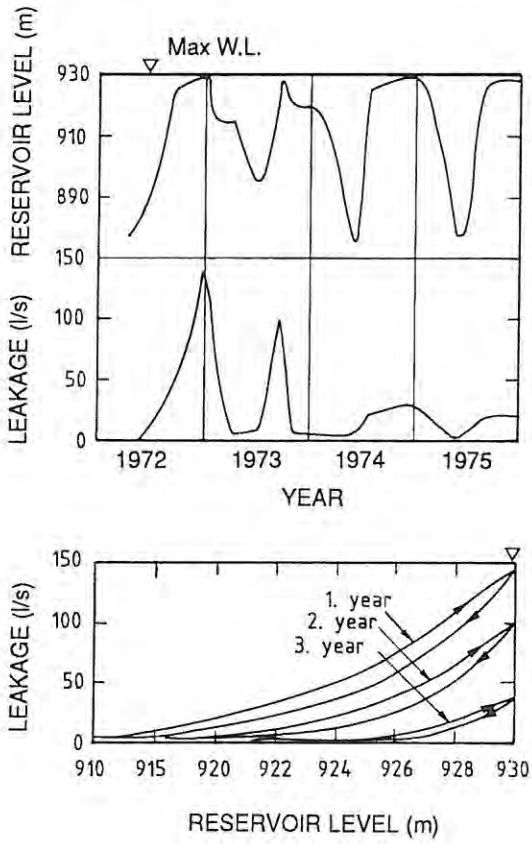


Fig. 8.1
Normal rate of seepage in relation to time and reservoir level

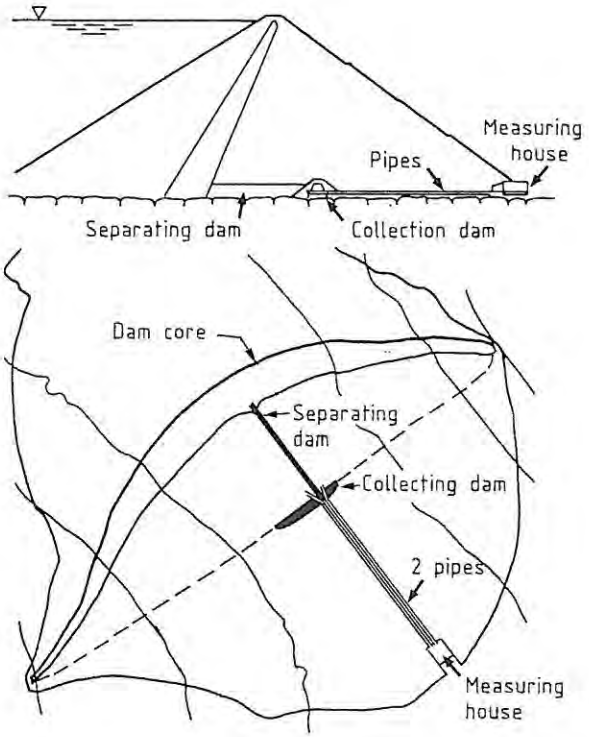


Fig. 8.2
Layout of facilities for seepage collection and measurement at Svartevann Dam

Water is collected separately for the right and the left halves of the dam and conducted through two pipes to a measurement chamber at the downstream toe. In the chamber the seepage water passes over a weir (Figure 8.3a), and the quantity of seepage is determined by the water head above the point of the V-notch.

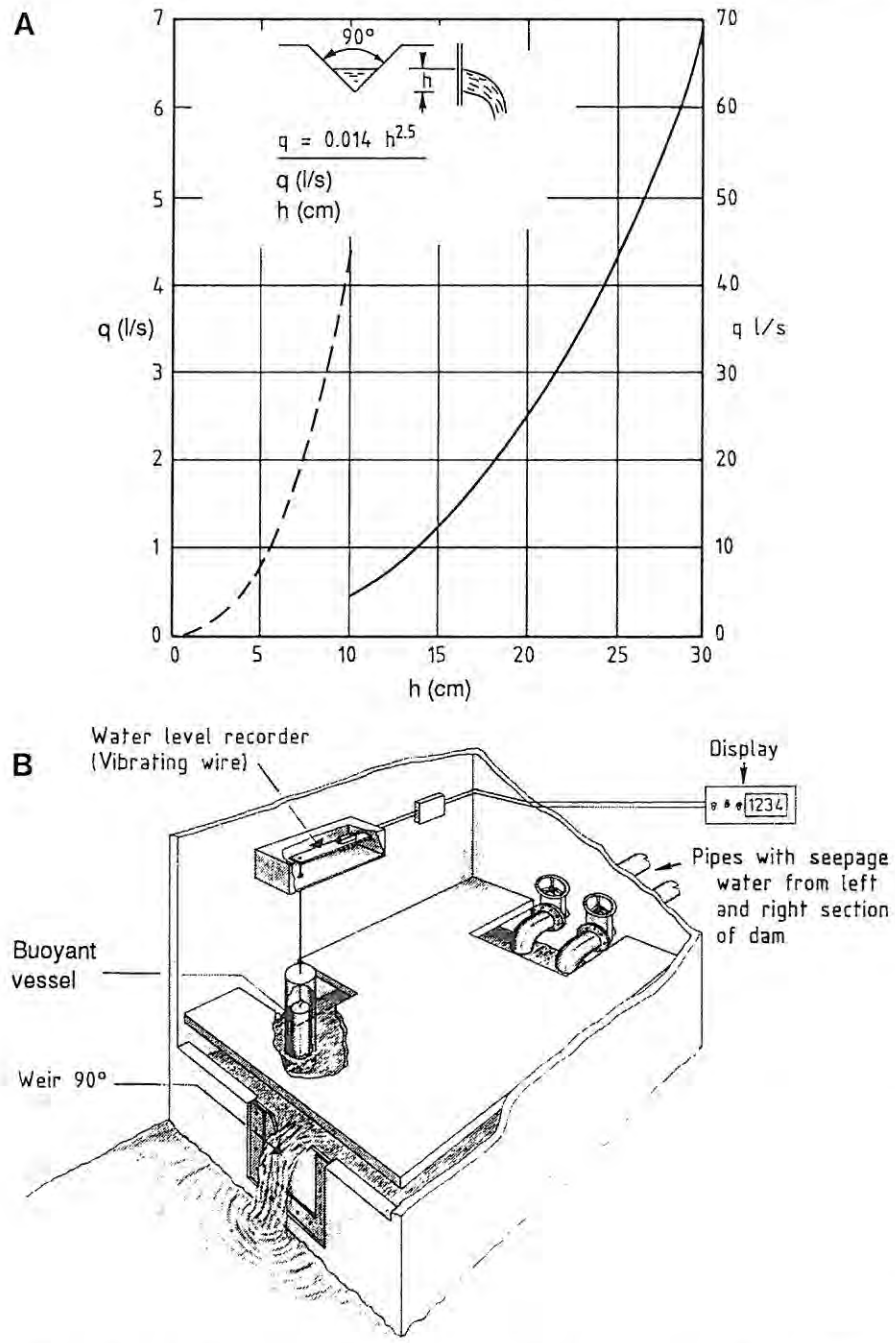


Fig. 8.3
Seepage instrumentation
A) The rate of seepage is measured by a V-notch weir
B) Automatic measurement of the head above a V-notch weir

The head may be measured manually by a fixed, calibrated scale. It can also be measured automatically by a buoyancy vessel, as shown schematically in Figure 8.3b. The electronic signal from the automatic readout instrument may be read remotely, for example via a telephone line. Remote reading of seepage is a necessity in many places, especially during the winter season.

8.2 Surface Displacements

The core is constructed with a freeboard above the maximum water level and with a camber to compensate for post-construction settlements. In order to verify, at any time, the elevation of the top of the core, settlement markers are placed as shown in Figure 8.4. Settlements of the markers and thus of the top of the core are determined by levelling.

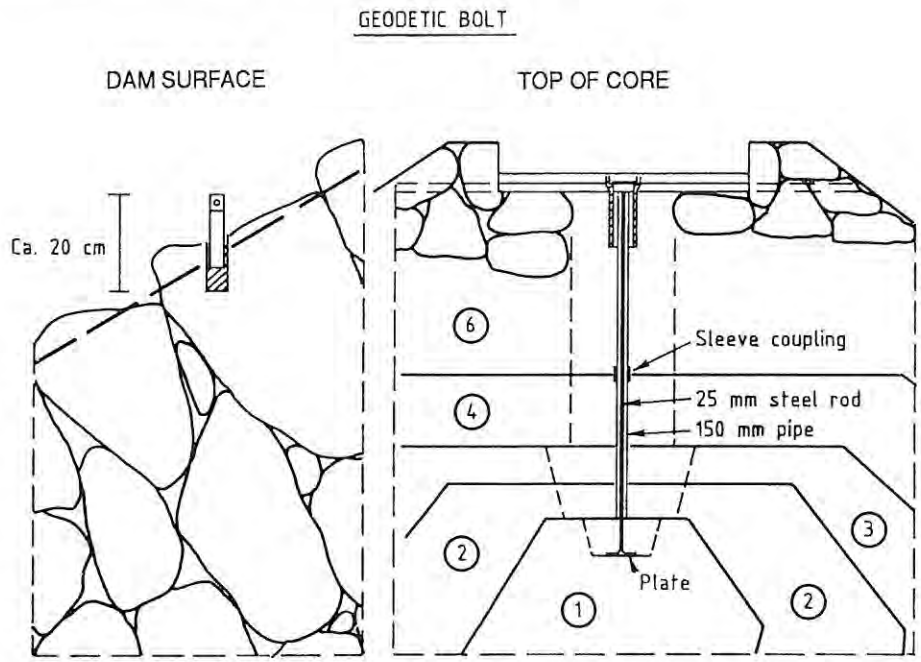


Fig. 8.4
Markers for measurement of surface displacements and top of core settlements

In order to observe what deformations are taking place within the dam, a network of markers is placed on the surface. The markers are installed and the coordinates determined by triangulation immediately after the embankment has reached the elevation for each horizontal line of markers.

The markers are grouted into holes drilled in large block stones firmly embedded in the slope. Deviations of one metre from the planned position in the network are permitted to allow selection of the most favourable stone for installation of the marker. Four to six reference pillars are used for triangulation of the markers. They are positioned on firm ground to the side of the dam so that each marker can be viewed from at least two reference pillars. The reference pillars must form a stable support for the observation instrument without acting as foreign objects in the landscape. A design as shown in Figure 8.5 will be taken as a cairn, at least viewed from some distance.

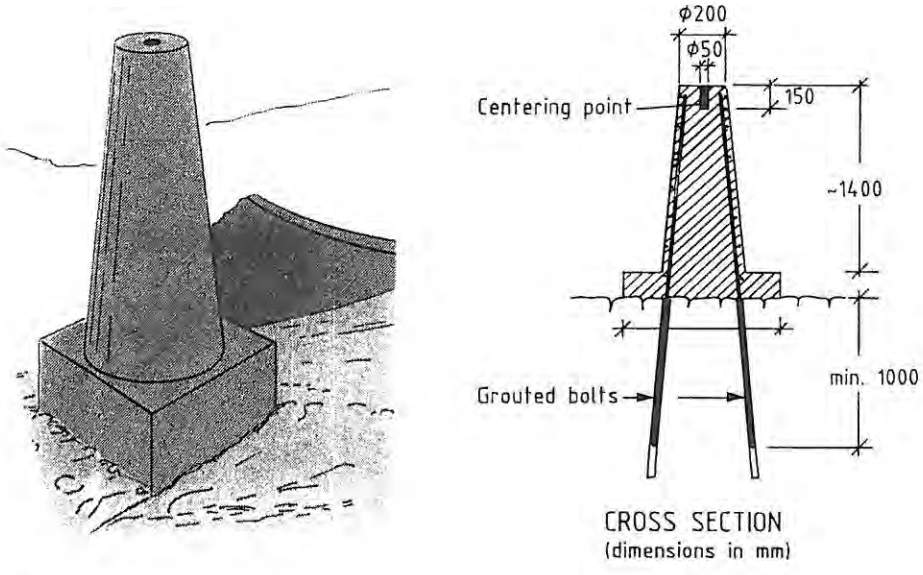


Fig. 8.5
Reference pillar for triangulation of marker bolts on a dam

An example of a network of marker bolts and reference pillars is shown in Figure 8.6. Contours of displacements on the surface of Svartevann Dam are shown in Figure 5.5, Chapter 5. The displacement of each marker bolt is recorded from the time of installation of each bolt. Thus at the time of completion of the dam, the displacement is recorded as zero at top of the dam and is having a maximum at about midheight.

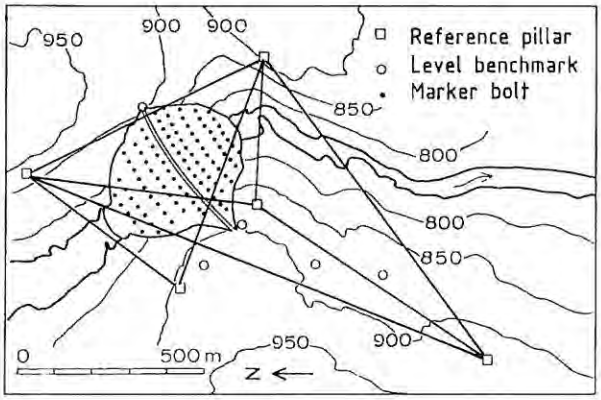


Fig. 8.6
Network of marker bolts and reference pillars at Svartevann Dam

8.3 Internal Displacements

By installing telescopic tube systems, vertical settlements may be measured at several levels within the embankment. The observations can include both settlement during construction, resulting from the added weight of the overlying fill, due to impoundment, and creep taking place over time during operation of the dam. The measurements are performed by lowering a sensor through the tube system and finding the depth to markers anchored to the fill at different levels. The compression of the fill between each marker is obtained and expressed as change in the distance between each pair of observations.

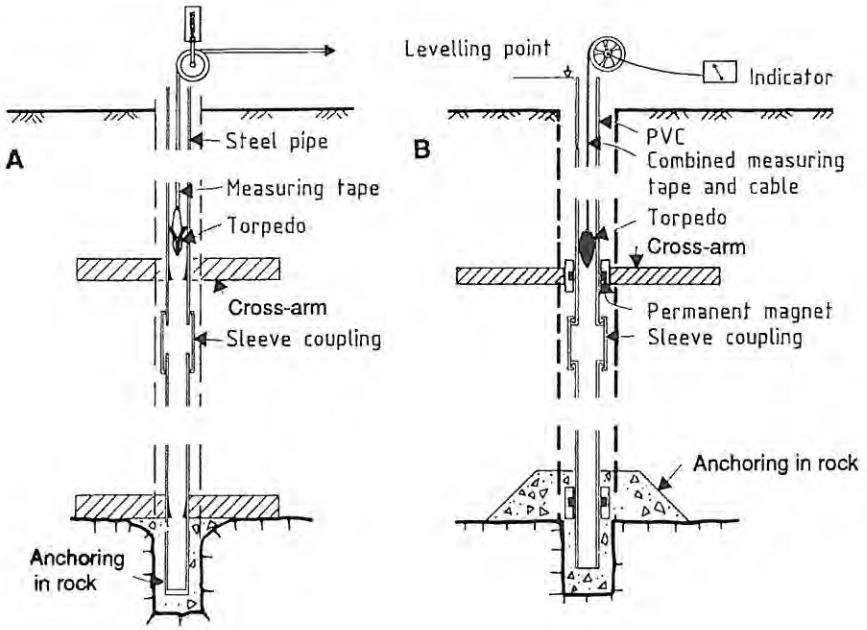
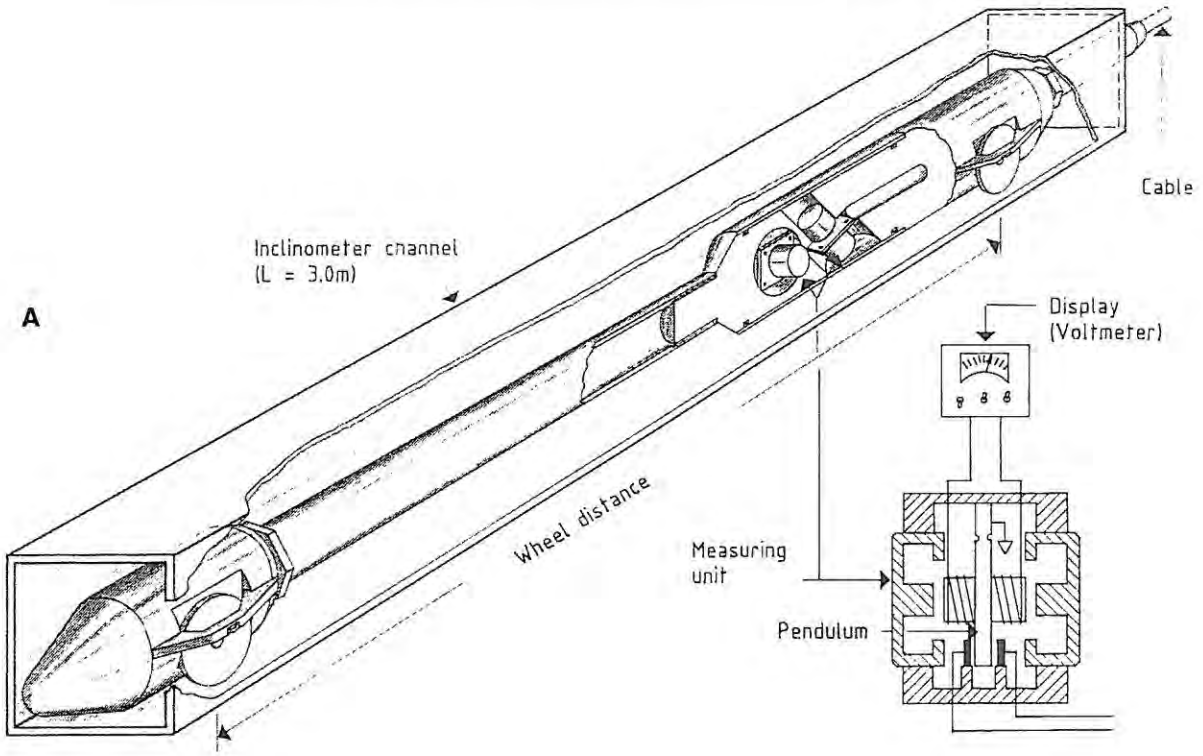


Fig. 8.7
Installation for measurements of vertical compression
A) Cross-arm system
B) Permanent magnets and a reed relay sensor system

At the Venemo Dam this type of measurement was carried out by the so-called "cross-arm" system. Figure 8.7 shows the principle of the tube



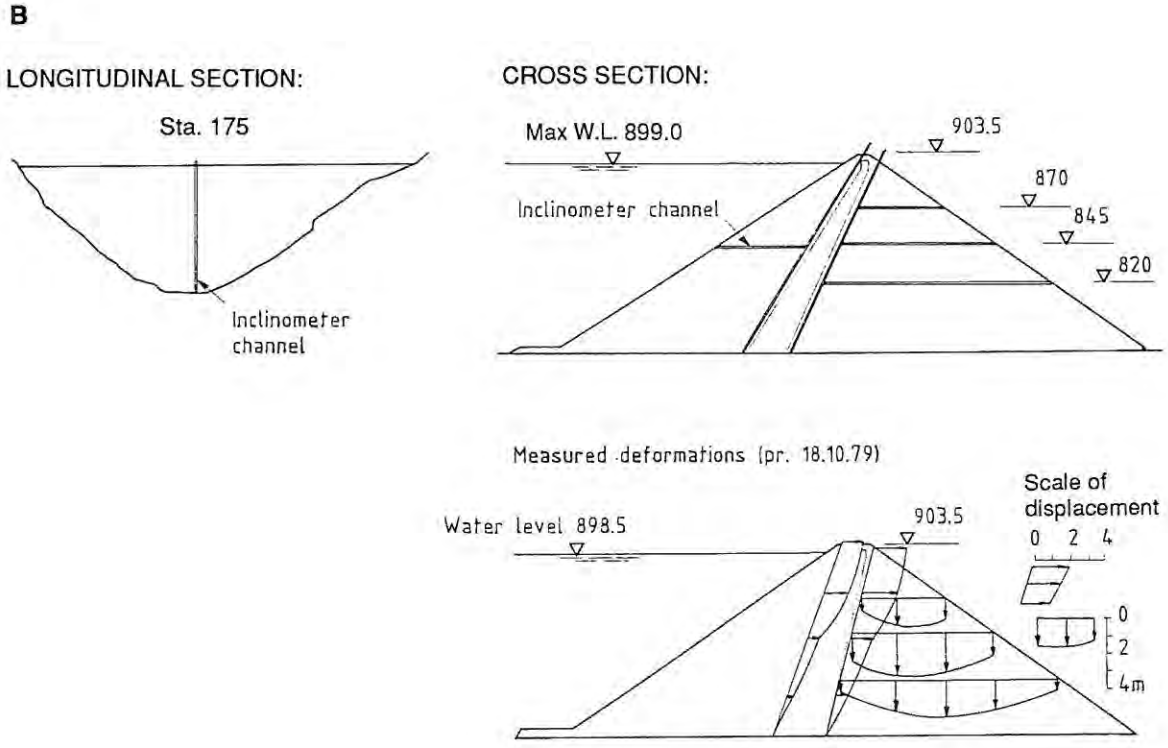


Fig. 8.8
Inclinometer instrumentation
A) Square inclinometer casing and inclinometer instrument with details of sensor
B) Inclinometer casings in the centre section of Svartevann Dam, installation and results

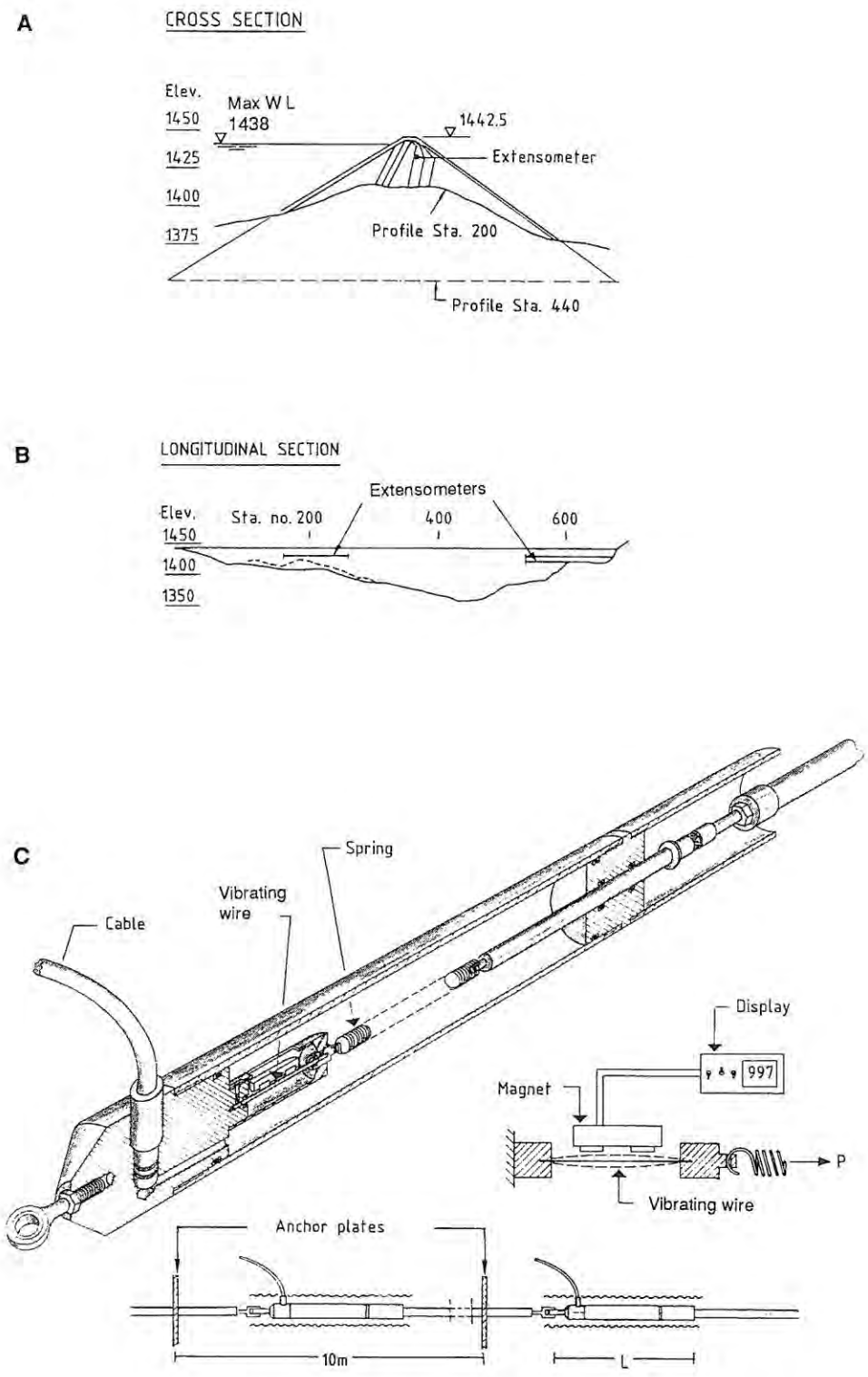


Fig. 8.9
Extensometer installation in Nyheller Dam
A) Cross-section
B) Longitudinal section, centerline core
C) Extensometer details and principle of installation in a chain

system and, schematically, how the measurements are carried out. Alternatively, the "cross-arms" and latching arm torpedo may be replaced by a system with permanent magnetic markers and a reed relay sensor, as shown in Figure 8.7b.

By installing a flexible casing system in an embankment, the flexure and transverse displacement of the casing can be determined by an inclinometer (Figure 8.8). The casing system can be made from square sections or circular sections with internal grooves. In either case a plane is defined along which the inclinometer is guided. The inclination of the casing is usually measured in steps equal to the wheel spacing of the inclinometer. By integration of the offset in each step, the coordinates relative to the fixed casing end may be calculated. The displacements of the channel and thus the fill are obtained and expressed as changes in relative coordinates between each observation.

An inclinometer casing may be installed at any inclination. For example at Svartevann Dam, there are two inclining and four horizontal casings in one instrumented section (Figure 8.8b).

The two inclining casings are installed one at each boundary of the core. The bottom end is embedded in the rock foundation. The observations from these two casings give the upstream/downstream displacements of the core during construction and operation of the dam. The difference between the two casing deflections represents the change in width of the core. The four horizontal casings record the settlements inside the embankment relative to the outer ends which are positioned by triangulation.

Horizontal inclinometer casings may be combined with a system of plates for accurate measurement of horizontal displacements inside the embankment as described by Penman and Charles (1985). Each gauge is terminated in a small instrument house built on the downstream slope.

Parts of an embankment dam can experience extensional strains as discussed in Chapter 5. These strains may cause cracking both parallel and transverse to the dam axis. Extensional strains often develop near the top of the dam close to the abutments, and unfavourable topographical conditions in the foundation can also cause extensional strains.

The longitudinal displacements resulting from extension can be measured by extensometers. Typically extensometers are installed horizontally and parallel to the dam axis. An example from Nyheller Dam is shown in Figure 8.9. The extensometers are installed in chains at several levels in the dam profile. If large leakages occur, the extensometer observations may help to locate the source of the leakage.

8.4 Pore Pressure Measurements

For dams founded on soil overburden or fractured rock, observations of pore pressures in the foundation will be important for the evaluation of dam safety. The measurements should include the substrata beneath and down-

stream of the dam as well as the lateral abutments. Pore pressure instruments in a dam foundation are most often installed in boreholes from the ground surface to the desired measurement depth (Figure 8.10).

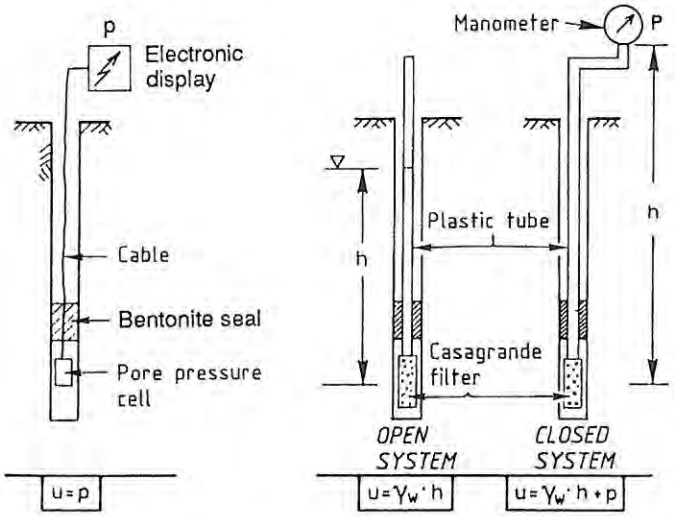


Fig. 8.10 Principle for measurement of pore pressure in boreholes

The instrument may be an electronic sensor, a pneumatic sensor or a Casagrande type filter. From an electronic sensor a cable is run up the borehole to a readout instrument. A Casagrande filter is connected to a plastic tube through which the water may rise to an elevation corresponding to the pore pressure in the ground around the filter. The water level in the tube is determined by an electrical sensor lowered down the tube. If the water level rises above the top of the tube, the excess pressure is measured by manometer. Sometimes several instruments are mounted in one borehole.

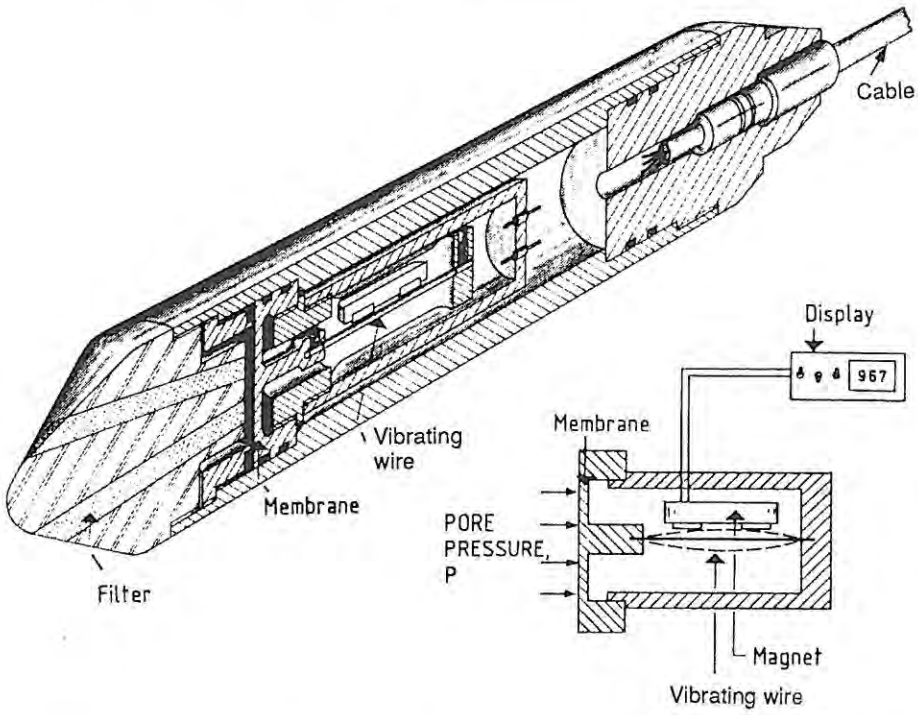


Fig. 8.11 Electrical piezometer for pore pressure measurement

Hydraulic, pneumatic or electronic sensors may also be placed inside the dam where pore pressures are to be observed. From the sensors or the filters, cables or tubes are run to an observation chamber. This is often com-

bined with the seepage observation chamber. An electrical piezometer for pore pressure measurement is shown in Figure 8.11.

Pore pressure measurements inside an embankment dam are used primarily to observe pore pressures during construction and are utilized as a part of the construction control. It may also be of interest to observe pore pressures at full reservoir and during rapid drawdown. The observations may be used to check seepage analyses carried out for the design of the dam and any change with time of pore pressures and hydraulic gradients in the core.

Practical and economic factors limit the number of pore pressure instruments, and they must be concentrated in one or a small number of instrumented cross-sections. The sections are generally selected to give a representative, overall picture of the pore pressures in the dam. Thus, the pore pressure observations rarely indicate any local deviation or impairment in the dam.

8.5 Total Stress Measurements

Earth stress cells may be installed in the body of the dam to measure the total stresses acting on different planes. However, it is difficult to obtain a true measurement of stress inside a soil mass. This is because the installation of the stress cell itself and the different rigidity of the cell and the soil it replaces may change the soil stress at the point of interest. Experience shows that it is simpler to get reliable measurement of soil stress on the interface between a structure and soil, e.g. against an abutment wall.

An earth stress cell is illustrated schematically in Figure 8.12. Most earth stress cells measure the total normal stress on the sensitive plane, i.e. the sum of the effective stress and the pore pressure.

Observations of soil stresses are of special interest for evaluation of the cracking potential. Properly positioned, stress cells may be used in combination with extensometers. Figure 8.12 illustrates an example from Akersvatn Dam where earth stress cells were used to evaluate the potential for hydraulic fracturing. The right hand part of the figure shows the instrumented section and the left hand part some typical results.

The measurements show that the weight of the core is partly transferred to neighbouring filter zones (arching). In this way the soil stress can be less than the water pressure at full reservoir, and hydraulic fracturing may occur (Chapter 5). As a result the rate of seepage can suddenly increase and a leak with internal erosion can develop unless the core and filter design leads to selfhealing (Section 2.3).

It should be observed that earth stress cells installed in the core are often displaced and rotated during placement and compaction of the core material above the cells. To overcome this problem, stress cells are sometimes equipped with an inclinometer to observe its true orientation in space (DiBiagio et al., 1982).

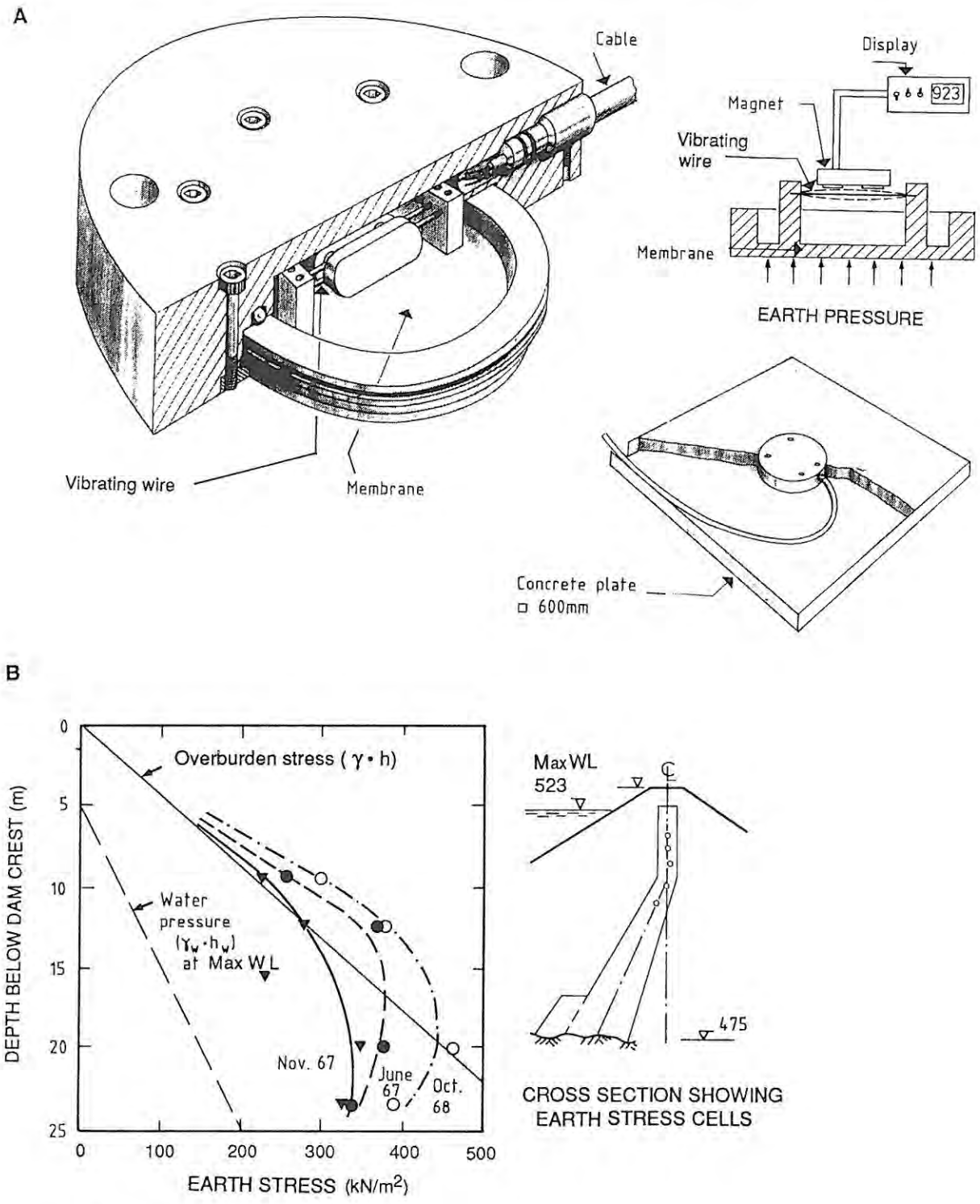


Fig. 8.12 Total stress measurements

A) Earth stress cell for measurement of total stress in embankment

B) Measured stresses in the core of the Akersvatn Dam shortly after completion of construction

8.6 Recommendations

A plan for instrumentation and performance observations should be incorporated in the design of a dam. The minimum instrumentation plan must include installations for measuring seepage and settlements at the top of the dam. Instrumentation beyond the minimum plan will depend on special features of the project. The primary purpose of the required measurements will determine the extent and position of installations needed.

Instrument installations in an embankment dam have to be compatible with construction procedures, as well as to satisfy the measurement requirements. Different types of instrumentation should be combined so that observations supplement one another. In this way, possible malfunctions can be checked and verified against other instruments. Obvious errors may be eliminated and data supplemented for instruments that are malfunctioning. The installations should be put in pairs so as to increase the redundancy of the instrumentation in places of special importance or when using sensitive instruments.

Even a modest instrumentation programme will generate a considerable amount of data over time. Processing the data should be done by computer, and the necessary records may be recalled and presented in clear diagrams for each instrument or for each set of instruments by employing systematic storage in a data base. The diagrams may present all the available data or they may concentrate on data of special interest.

After the first filling of the reservoir the responsibility for monitoring and maintenance of the dam is often transferred to an organization other than that which designed and constructed the dam. On transferring responsibility, special attention should be paid to future requirements for evaluation of the data.

Observations from instrumented dams represent full scale test results. Instrumentation costs beyond the safety needs of the individual project are often considered superfluous. The observations will, however, generate increased knowledge about the behaviour of embankment dams in general. Insight into actual performance represents important experience for further development of the state of the art from which future projects will benefit.

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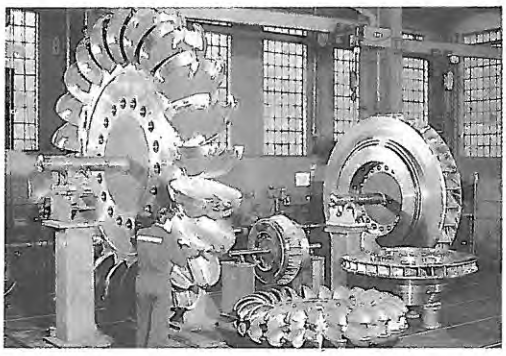
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The Power Division is a division within Siemens A/S. Siemens A/S ranks as one of the leading electro-technical specialists in Norway, with some 4000 employees and a 1989 turnover exceeding NOK 3.6 billion.

Export over the years from the Power Division include a variety of products and systems, ranging from 12, 24 and 36 kV outdoor isolators to complete 12 and 24 kV sub-stations for Scandinavia.

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